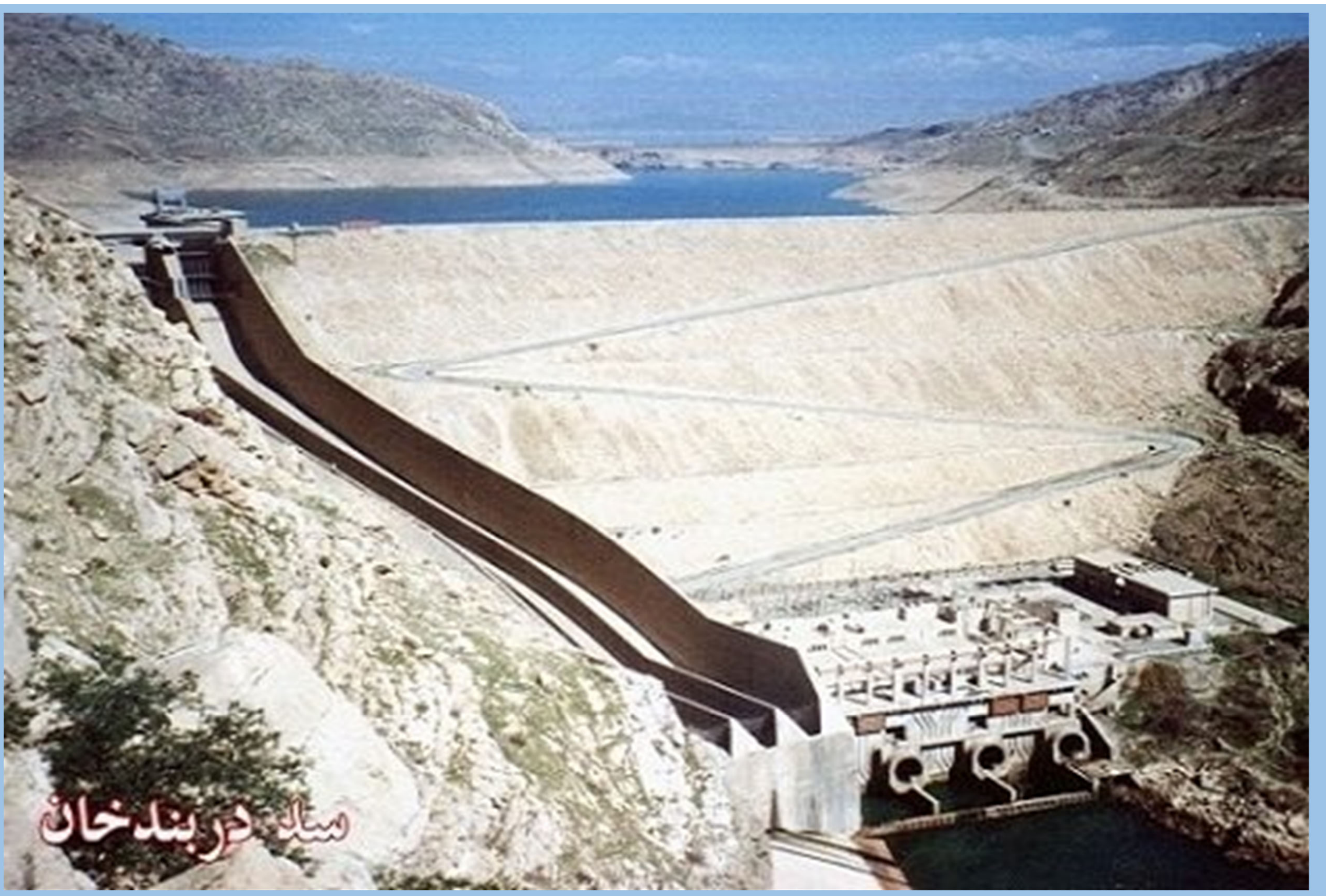

Dam Safety Review: General Principles and Procedures. Part 2

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Author's Note

This book has been prepared and published in two Parts as collection of separate papers on issues related to Dams Safety to contain as much information on this topic as possible. It provides a review of current practices on the subject together with actual accidents and failures due to natural causes, failure of designers to recognise proper safety requirements or operators' short sights in operation or maintenance. The author wishes to extend his warmest thanks to Mr. Varoujan Sissakian for the editing of the manuscripts of all the papers and to Professors Nadhir al- Ansari, Jan Laue and Sven Knutsson from Lulea University of Technology for the generous grant offered by the university for the publication.

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Dame Safety: The question of Tailings Dams

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Abstract

Tailings' Dams are mining waste impounding structures. They differ from conventional dams in purpose, design and operation. Percentage wise their failures are higher and posing considerably more safety concerns, causing long lasting devastation on communities, environment, and animal and plant ecosystem. Two basic types of embankment tailings structures are used for tailings impoundments; the Retention Dams and the Raised Dams. Retention dams are built in one operation to a full height, while construction of Raised Dams is a continuous process lasting for the whole useful life of the mine. Raised Dams are favored over Retention dams as they can be enlarged and expanded as the extraction works continue with time. Raised embankment dams themselves can be of three alternative designs according to the method used in construction; the Downstream, Upstream and Centerline structures. This designates the direction in which the embankment crest moves in relation to the initial embankment at the base as successive lifts are added. Resulting from the used method of tailings weight disposition, the Upstream Raised Dams are the least safe in earthquake prone areas as compared to the other two types due to its higher possibility of liquefaction, so they are not favored in highly seismic areas. The disadvantage of Downstream Raised Dams is their use of larger land areas. Centerline Raised Dams are a compromise between the other two. Tailing Dams failure may occur due to: dam instability, overtopping, internal erosion, or combination of these. Instability can result from faulty design and/ or faulty tailings deposition method. Internal erosion can follow saturation of the fill due to fast rate of work and close proximity of the water pond to the dyke combined with downstream gullying, and overtopping happens in case of faulty water management and/ or inoperable decan system. Careful analysis of historic failures and drawing

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out new lessons from them can help reducing failure probability and enhance tailings' dams' safety.

Keywords: Tailings, Tailings Dams, Mining, Retention Dams, Raised Dams, Downstream Structure. Upstream Structure, Centerline Structure, Dam Instability, Overtopping, Internal erosion, Decant System

1. General

Tailings Dam is typically an earth-fill embankment dam used to store commercially worthless material that surrounds, or is closely mixed with, a wanted mineral in mining operations after separating the ore. Tailings themselves can be liquid, solid, or a fluid of fine particles, and are usually highly toxic and potentially radioactive. Solid tailings are often used as part of the dam structure itself. The contents of the tailings depend on what mineral is being mined. More than half of the material removed from mines is cast away immediately, then only the ore-containing the required material is processed, by grinding it up to very small particles, and then using some chemical additives to extract the wanted material.

Normally, the salvaged material is less than 1% of the original material put through the processing. So, this usually ends up with a lot of waste. The waste often is in the form of a slurry: a fairly even combination of solid waste and water. A slurry's specific contents vary based on the type of mining being done and where the mine is located [1].

Many of the mined minerals are called sulfide minerals. The most common sulfide mineral is the iron sulfide "pyrite". So, when pyrite is exposed to oxygen and to water it breaks down chemically into a weak sulfuric acid. That acid, in turn, will dissolve some of these other accompanying sulfide minerals that contain lead, zinc, cadmium, selenium, arsenic, and a whole host of other things that can be dangerous for the environment.

Mining of copper, gold, uranium and other mining operations similarly produces various kinds of waste; much of it is toxic or radioactive, which pose varied challenges for long-term containment. Traditionally, around 90% of gold is processed using hazardous chemicals like cyanide or mercury. Using these toxic chemicals to release fine gold ores from rock is a process that has existed for over 100 years. These toxic chemicals are often expelled into tailing dams with waste water from the extraction process.

Even if the dam does not collapse, these highly poisonous substances can still leach into the local surrounds, or ground water contaminating the soil and water supply. Dam's collapsing, however, means not only overflooding large areas and the possible destruction of communities causing human fatalities, but also spreading toxic materials over extensive terrain with long lasting damage inflicted on the environment. For a person, just a grain-sized dose of cyanide or 1.5 grams of mercury is deadly. For fish, trace amounts of cyanide would prove fatal. This means that tailing's dams' safety is a paramount matter that should be taken seriously by all concerned.

Tailings' Dams rank among the largest engineered structures on earth. The Syncrude Mildred Lake Tailings Dyke in Alberta, Canada, is an embankment dam about 18 km long and between 40 to 88 m high. It is the largest dam structure on earth by volume fill, as of 2001. It is located 40 km north of Fort McMurray, Alberta, at the northern end of the Mildred Lake. The dam and the tailings artificial lake within it are constructed and maintained as part of ongoing operations in extracting oil from the Athabasca oil sands. Other tailings' dams constructed and operated in the same area include the Southwest Sand Storage (SWSS), which is the third largest dam in the world by volume of construction material after the Tarbela Dam. The facility was designed to provide coarse tailings sand storage, returning water and thin fine tailings to other sites within the Mildred Lake Project area [2].

Tailings' Dams are much different from the familiar conventional dams which serve the multipurpose objectives of flood control, storage of water for agriculture and drinking water in addition to power generation. Tailings dams are meant for permanent storage of unwanted materials and have different design and methods of construction. As such Tailings Dams present different challenges to the geotechnical engineers than those they face with conventional dams.

Facts that need to be recognized at the start of any Tailings Dam developments are: i) These dams must remain structurally competent in "perpetuity" (perpetuity is a long time) and cannot be breached upon project completion, ii) They are constantly changing in size and often reaching hundreds of millions or, occasionally exceeding one billion of tons of material to utilize or store, iii) They are in ever changing states of stress, iv) Typically, under construction for at least 5 to 10 years, but construction can be extended to periods of more than 50 or even 100 years, v) They are susceptible to brittle undrained loading response, vi) Tailings Dams have no ability to generate revenue for their owner, as opposed to hydro-electric dams for example, and so generally thought of as burdens rather than assets and an annoying way of doing costly business, and vii) They seldom have owners who are familiar with all the key geotechnical issues facing these facilities and thus putting such responsibility on the consulting designer [3]

2. Tailings Dams Failures and Consequences

It is estimated from information available in 2000, there were at least 3,500 tailings dams around the world. But, as there are around 30,000 industrial mines, the number of tailings dams is likely to be much higher [4]. This number is not confirmed due to the unavailability of complete inventory and the total number is disputed. These structures experience known "major" failures of about 2 to 5 annually, along with 35 "minor" failures. Assuming the 3,500 figure is correct, this failure rate is more than two orders of magnitude higher than the failure rate of conventional water retention dams.

In one publication from 2001 of ICOLD- UNEP, it estimated the number of major incidents with Tailings Dams were continuing at an average of more than one per year. During the 6 years previous to that, the rate had been two per year. Trying to determine the causes of these incidents, 221 case records were collected. The main causes of the reported failures and incidents cases were found to be: 1) loss of control over water balance, 2) lack of control over construction and 3) a general lack of understanding of the features that govern safe operations. There were one or two cases of unpredictable events and other cases caused by unexpected climatic and natural conditions, including earthquakes [5].

List of Tailings Dams failure, prepared by the World Information Service on Energy Projects (wise) and updated on 29th December 2019, documents 126 failures of various sizes and types of Tailings Dams for the period 1961- 2019 giving an average of about two cases per year. But due to limited availability of data, this compilation is considered in no way complete.

An abridged list drawn by the writer from (wise) list is presented in Table (1), showing only some arbitrary selected cases to indicate countries where major failures occurred, and illustrate a type of mining operations and the type of extracted ore, in addition to size and extent of the resulting damage [6].

Table 1: Chronology of major tailings dam failures (1961- 2019)

Date	Location	Ore Type	Incident Type	
2019,July 10	Peru, Corbiza mine	Copper	Tailings Dam Failure	67, 488 m3 tailing release, covering 41,574 m2
2019,Jan. 25	Brazil, Córrego de Fejão mine	Iron	Tailings Dam Failure	12 million m3 of tailings wave travelling 7 km destroying infra structures killing 259 peoples
2018, Mar. 9	N. S. W. Australia, Cadia	Gold, copper	Tailing Dam Failure	1.33 million tons of tailings . Failure due to low density foundation layer in the vicinity of slump
2016,Aug.27	USA, Florida, New Wales Plant	Phosphate	Sinkhole in the phosphate-gypsum stack	840,000m3 of liquid released as of Sep 2016 contaminating a major the Floridan aquifer ; a

				major source of drinking water
2015, Nov.5	Brazil, Germano mine	Iron	Tailings Dam Failure due to insufficient drainage leading to liquefaction after a small earthquake	32 million m3 slurry wave causing 158 homes destroyed, 17 persons killed in nearby town of Bento Rodrigues. Polluting North Gualaxo River, Carmel River and Doce
2014, Aug.4	British Colombia, Mount Polley mine	Copper, Gold	Tailings Dam Failure due to foundation failure	7.3 million m3 of tailings, 10.6 million m3 of water and 6.5 m3 of interstitial water flowing into Polley Lake and, through Hazeltine Creek into Quesnel Lake
2012, Aug.1	Philippines, Padcal mine	Copper, gold	Tailings Pond breach	20.6 million tons of tailings discharged into Balog River
2009,Aug.29	Russia, Karamaken Mineral Processing plant	Gold	Tailings Dam Failure after heavy rain	More than 1 million m3 of water,150.000 m3 of tailings and 55.000 m3 of dam material. eleven homes washed away by mud flow, one person killed
2007, Jan 10	Brazil, Mirai	Bauxite	Tailings Dam Failure	2 million m3 of mud of clay and water left 4000 residents homeless, crops and pastures destroyed, water supply compromised

2005,April.14	USA, Mississippi, Jackson county	Phosphates	Phospho-gypsum stack failure	17 million gallon of acidic liquid (64,350 m3). Liquid poured into adjacent marsh lands
2004, Sep. 5	USA, Florida, Riverview	Phosphates	Dike on 100 gypsum stack holding 150 million gallons of polluted water broke after waves driven by Hurricane Frances bashed the dike	60 million gallons (227, 000 m3) acidic liquid spilled into Archie Creek leading to Hillsborough Bay
2000, Oct.11	USA, Kentucky, Inzes	Coal	Tailings Dam Failure	250 million gallons(950.000 m3) of coal waste. 120 km of rivers turned iridescent black casing fish kill
2000, Sep. 8	Sweden, Aitiki mine	Coper	Tailings Dam Failure	2.5 million m3 of liquid released into settling pond and releasing 1.5 million m3 of slurry into adjacent environment
1995,Aug.19	Guyana, Omai	Gold	Tailings Dam Failure	4.2 million m3 of cyanide slurry causing environmental disaster
1994, Nov.19	USA, Florida Hopewell mine	Phosphate	Tailings Dam Failure	1.9 million m3of water from clay settling pond spill into nearby wetland and Alafia

				River, Keysville flooded
1994, Oct. 2	USA, Florida, Pine Creek mine	Phosphate	Tailings Dam Failure	6.8 million of water from clay settling pond. 500.00 m3 released into Hickey Branch a tributary of Payne Creek
1994, Feb. 14	South Australia, Olympic Dam, Roxby Downs	Copper, Uranium	Tailings Dam Leakage	5 million m3 release of contaminated water into subsoil
1992, Jan.	Philippines, Padcal mine	Copper	Collapse of dam wall	80 million tons…?
1982, Nov. 8	Philippines, Spialay	Copper	Tailings Dam Failure	28 million tons inundation of agricultural land up to 1.5 m depth
1980, Oct. 13	USA, New Mexico, Tyrone	Copper	Tailings Dam Breach	2 million m3 tailings flow 8 km inundating farmland
1974, Nov.11	South Africa, Bafokeng	Platinum	Embankment Failure	3 million m3 , 12 people killed, tailings flow travelled 45 km
1965,Mar. 28	Chile, El Cobre , New and Od Dams	Copper	Liquefaction dam failure during earthquake	350,000 m3 and 1.9 million m3, tailings travelled 12 km killing more than 200 people
1962, Sep. 26	China, Huogudi, Gejiu, Yunnan	Tin	Failure of upstream dam due to moderate rain	Tailings travelled 4.5 km, destroying 11 villages, 171 people killed, 92 injured 13,970 homeless

Failure of tailings storage facilities (TSF) can have disastrous consequences for nearby communities, the environment, and for the mining companies, who may consequently, face high financial and reputational costs. In 2015, the breach of the Fundão Tailings Dam at Samarco mine in Minas Gerais resulted in 19 fatalities, and was declared the worst environmental disaster in Brazil's history. In settlement of the claims resulting from this failure, the owners had to agree on a settlement plan costing $2.4 billion to remediate and compensate for the impacts over a 15 years; moreover, twenty-one company executives were charged with qualified murder. The ecosystems impacts caused by any failure can last for many years depending on the nature of the tailings. In the above mentioned example, the owning company is in the process of restoring 5000 streams, 16,000 hectares of "Permanent Conservation Areas" along the Doce River basin, and1200 hectares in the riverbanks. It is estimated that the livelihoods of more than 1 million people were affected because of the failure.

Improvements in the design, monitoring, management, and risk analysis of Tailings Dams facilities are needed to prevent future failures and mitigate the consequences of a breach [7].

3. Tailings Dams Types and Their Construction Methods- Their Pros and Cons

Two basic types of embankment structures are used for retaining tailings in impoundments; these are the Retention Dams and the Raised Dam. Retention dams are similar to conventional dams in regard to soil properties, surface water and ground water controls, and stability; they are built in one operation till completion to a full height at the beginning of the disposal operations. On the other hand, construction of Rising Tailings Dams is a continuous process which lasts for the whole useful life of the mining project depending on the size of the mine and its capacity. In both cases, suitable site is chosen first to contain the mining operations' wastes.

Typically for a rising tailing dam, a base or starter embankment is constructed first; this is done by surrounding the pre-selected terrain for the site by primary embankment, which may take the shape of a circular ring or any other shape depending on the topography of the terrain to form an enclosure. The embankment may be built of material consisting of natural soils, mine overburden, other current mine waste or tailings from a preexisting tailings deposits. Alternatively, and more commonly, the outer embankment is built of current tailings production. If the tailings comprise a wide range of particle sizes, it is a common practice to separate the sand (coarse fraction) from the slimes (fine fraction); first. The sand is used to build the outer embankment or shell of the impoundment, while the slimes are delivered into the body of the impoundment. When the tailings have a narrow range of particle sizes and the fine fraction is not clayey, the total product may be used to build the outer embankment. Since the object of a tailings dam is to retain solids, not water, the embankments are usually designed to be as pervious as possible [8].

An aerial view of Wheal Jane tin mine in Figure (1), is an example of tailings dam layout utilizing shape of the surrounding terrain [9]. This tailings dam was opened in mid-18th century and closed in 1992 in West Cornwall, England [10].

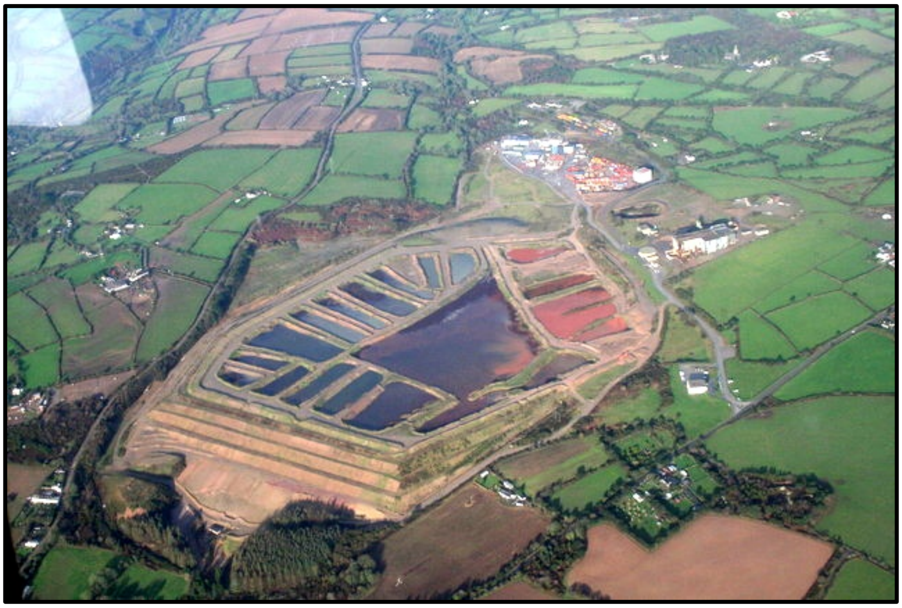

Figure 1: Aerial view of Wheal Jane tin mine Tailings Dam [9].

The enclosed area of impoundment may be one of four types; the Ring-Dike, In-Pit, Specially Dug Pit, and variations of the Valley design. The design choice is primarily dependent upon natural topography, site conditions, and economic factors. Most tailings dams in operation today are a form of the Valley design due to lesser volume of fill material and maximizing the use of local materials, particularly the tailings themselves which results in lesser overall cost. Tailings Retention Dams are suitable for any type of tailings and deposition method but commonly less used than the Raised Tailings Dams; typical cross section of such a type is shown in Figure (2) [11].

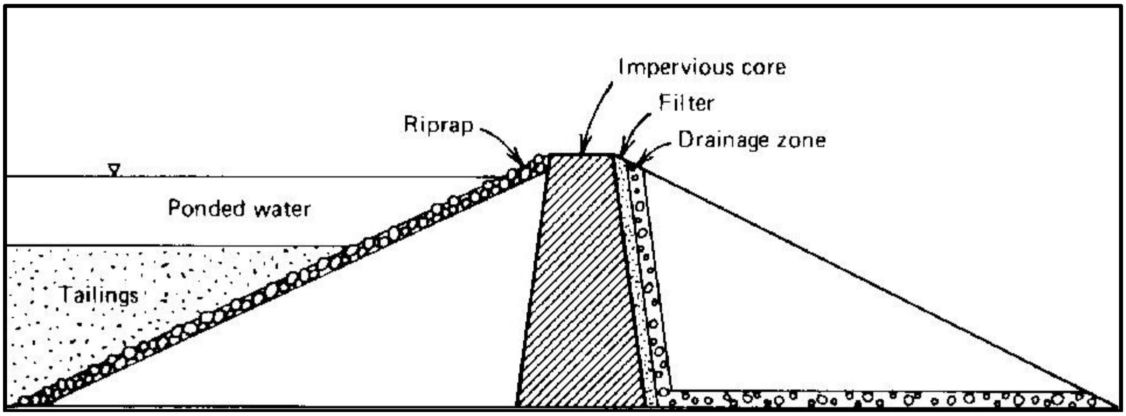

Figure 2: Typical cross section of Retention Type Dam for Tailings Disposal [11]

Raised Tailings Dams can be of three alternative designs according to the method used in construction; the Downstream, Upstream and Centerline Structures, which designate the direction in which the embankment crest moves in relation to the initial embankment at the base as the embankment is raised in successive lifts to increase the available volume for the storage of tailings and water.

The Upstream Method is the lowest initial cost alternative due to the minimal amount of fill material required for initial construction, and it is a more popular type of raised tailings dams in low risk seismic areas. The construction starts with a pervious starter dyke foundation, then tailings are discharged from the top of the dam crest creating a beach that becomes the foundation for future embankment raises. Figure (3) shows a diagram of the stages of construction of an upstream raised embankment.

By natural segregation, coarse material settles first and the fines furthest away. Cyclones can be used to accelerate this particle segregation for certain tailings by sending the slime proportion to the center of the impoundment and the sand fraction to the beach behind the crest.

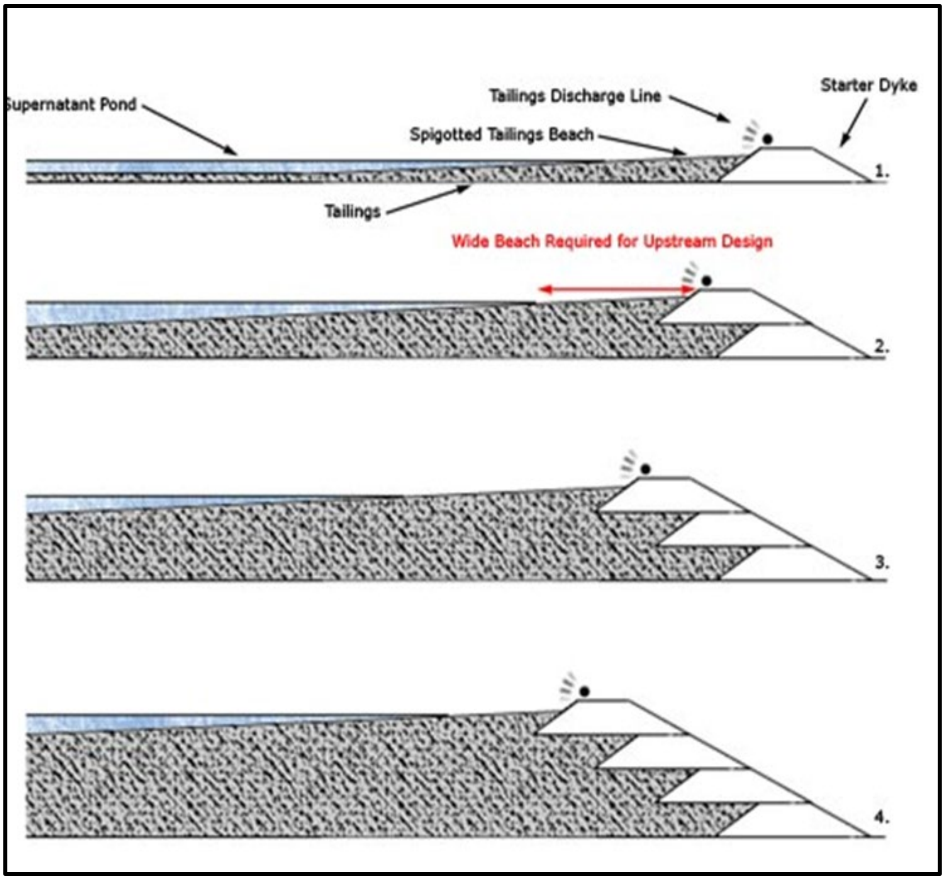

Figure 3. Upstream method of embankment construction

The conventional method of upstream raises relies on consolidation of the beach that forms the embankment shell under its self-weight, but today, compaction has become more common using earthmoving equipment to increase the degree of safety of raised embankments. Once the tailings have dried and consolidated, and tailings placed on the embankment crest and compressed, then tailings discharge lines are lifted and reassembled, and the next riser is started.

It is noted that the Upstream Tailings Dams failures are the more reported cases of failures causing huge environmental consequences all over the world. Records show that out of the just over 3500 tailings dams worldwide; 50% are of this design, and it was observed that the key failure mode of upstream embankments is a static/ transient load which induced liquefaction flow slide event. This is not surprising considering the low relative density of the tailings and the potential for water mismanagement to generate high saturation of the embankment and subsequently creating liquefaction induced flows of the tailings.

The high rate of failures of Upstream Tailings Dams has led to the development of the Downstream Embankment design to reduce the safety risks associated with the upstream design, particularly when subjected to be dynamic loading as a result of earthquake shaking. The installation of impervious cores and drainage zones can also allow the impoundment to hold a substantial volume of water directly against the upstream face of the embankment without jeopardizing stability. Construction of such a dam begins with an impervious starter dyke which is provided with internal drainage arrangement, unlike the upstream design that has a pervious starter dyke. The tailings are at first deposited behind the dyke and as the embankment is raised; the new section is constructed and supported on top of the downstream slope of the previous section. This shifts the centerline of the top of the dam downstream as the embankment stages are progressively raised, Figure (4).

An advantage to the downstream design is that the raised sections can be designed to be of variable porosity to tackle any problems with the phreatic surface of the embankment. This can be particularly useful where a processing plant has made changes to increase efficiency and as a result alter the tailings characteristics. This may result in pumping more water to the tailings facility or alter the drainage characteristics of the newly deposited tailings.

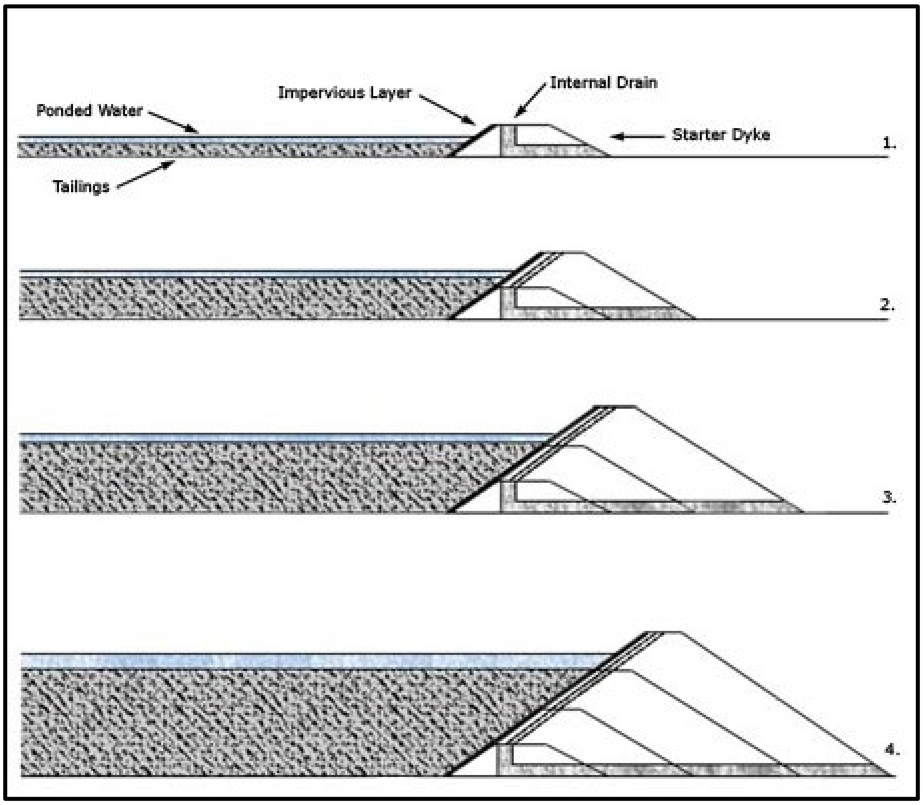

Figure 4: Downstream method of embankment construction

The downstream design is very versatile for a range of site specific design parameters and behaves similarly to water retention dams. Its main advantage is it can have unrestricted heights due to each raise being structurally independent of the tailings. The main disadvantage is the requirement of a large area around the dam itself as the toe of the dam moves out when more raises are added. This can cause problems where limited space is available, and can add up more cost for the acquisition of the required land area. Although a downstream embankment can theoretically have no height limit, the dam's ultimate height is determined by the restriction of the advancing toe. The raised dams are definitely more safe than impounded dams.

An improvement over two previous designs is the Centreline Type Tailings Dams. It is more stable than the upstream method, but does not require as much construction material as the downstream design. Like the upstream method, the tailings are generally discharged by spigots from the embankment crest to form a beach behind the dam wall. When subsequent raising is required, material is placed on both the tailings and the existing embankment. The embankment crest is being raised vertically and does not move in relation to the upstream and downstream directions of subsequent raises, hence the term, centreline design (Figure 5).

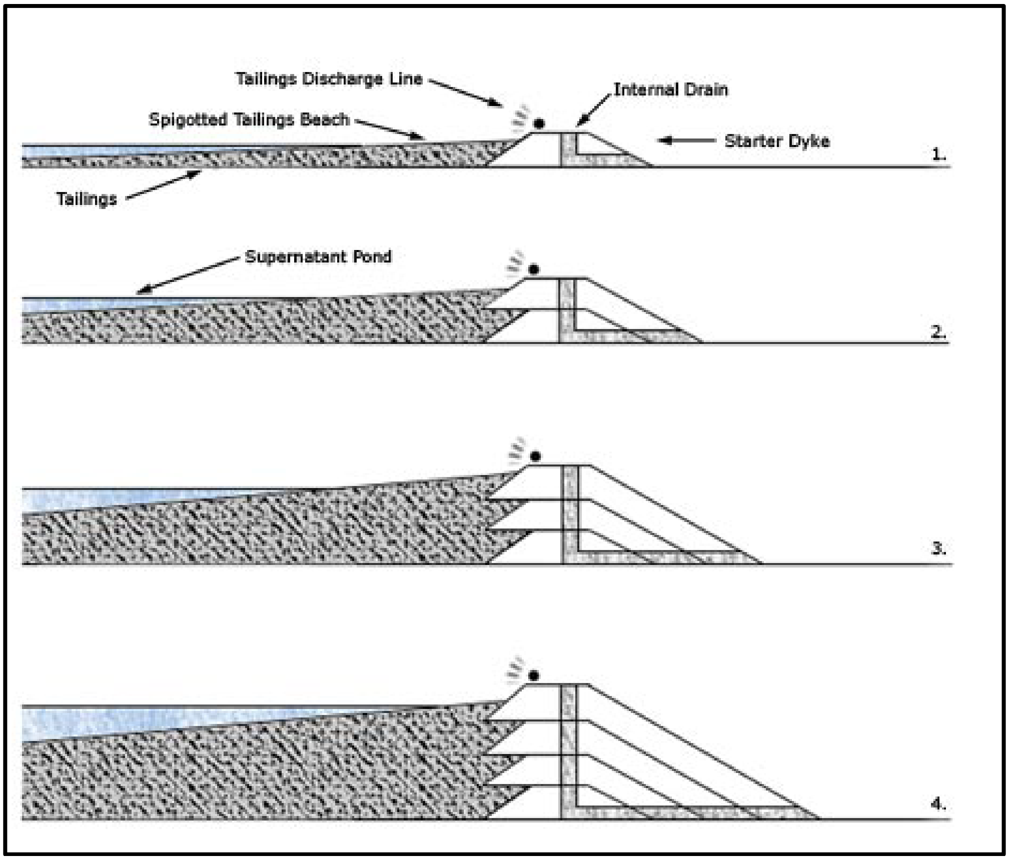

Figure 5: Centreline method of embankment construction.

The design incorporates the internal drainage zones that are similarly found in the downstream method. Therefore, the free water can encroach closer to the dam crest than the upstream method without the worry of increasing the phreatic surface and causing a potential failure risk. However, a Centreline Dam cannot be used as a large water retention facility solely due to the subsequent raises being partially built on consolidated tailings. A suitable decants emptying system needs to be installed to prevent the free water submerging the beach around the dam crest. In many cases, the centreline design is a good compromise between the seismic risk and the costs associated with construction. In a modification to this method is to reduce the volume of construction material placed in the downstream shell of the embankment, the angle of the upstream crest advance over the tailings is calculated during the design phase following stability and seepage analyses. Rockfill is usually utilized in this technique to gain a higher angle, rather than the coarse tailings fraction to reduce the risk of instability.

In countries where upstream construction is not permitted due to seismic risk, the modified centreline method may not be also permitted due to the concept of partially placing construction material on the existing tailings beach [12] and [13].

4. Impacts of Tailings Dam Collapse and Safety Hazards

The collapse of any tailings dam can have extensive negative impacts on the surrounding environment such as:

1) flooding and contaminating water streams and rivers,
2) in many instances even poison water supplies serving large populations,
3) covering large land areas with wastes rendering them unusable and very difficult and costly to reclaim,
4) crushing and destructing homes and infra structures
5) accumulation of poisonous residues in plants and animals, and 6) loss of animal lives and above all loss of human lives.

Failure of tailings dams may follow one of the following failure modes:

i) Dam Instability,
ii) Overtopping, and
iii) Internal erosion

Long-term safety hazards other than complete embankment failure can also result from such reasons as seepage, dust, long-term erosion. Tailings may retain their hazard potential for hundreds of years, which requires efficient measures to contain them in the long term.

From the report of the International Task Force for Assessing the Baia Mare and Baia Borsa Accidents in Rumania (January and March 2000) it can be seen that usually, there is a combination of reasons for the failure: The accidents were in summary caused by:

i) the use of an inappropriate design,
ii) the acceptance of that design by the permitting authorities; and

iii) inadequate monitoring and dam deficient construction in addition to operation and maintenance shortcomings.

Mentioned design and operational faults, which contributed to the failure were:

i) Use of a closed circuit system with no specific provision for the emergency discharge/storage of excess water,
ii) Inadequate care in the disposition of tailings within the dam and lack of homogeneity of the tailings
iii) Non operation of the hydro- cyclones in very low temperatures, and
iv) Failure to observe the design requirements for tailings grades for embankment wall construction [14].

Tailings dam modes of failures are classified into the different modes as already mentioned. More can be said on this as mentioned in the following three aspects [i) Dam Instability, ii) Water Management, and iii) Earthquakes]:

i. Dam Instability

Such instability has caused many tailings dams incidents, which were resulting from one or combination of the following four reasons:

a) Unsatisfactory foundations: Poor foundation means the soil or rock layer below the dam is not qualified to support the dam. This hazard can lead to partial or complete dam failure. This is allied to site investigation because the weaknesses revealed by the dam failures should have been detected during the site investigation. It does not necessarily imply that the site investigation was inadequate, but the dam designer may not have made the correct assumptions from the site investigation reports.

Complete and partial failure cases which were investigated by Karl Terzaghi and quoted by him include many examples of such failures: In the first, where the surface layers of clayey soil had not been stripped prior to the construction of the dam. The result was that the base of the dam slid forwards, causing failure. In a second case, an 8 m high tailings dam was built on gently sloping ground on a clay stratum about 6 m thick overlying a shale/mudstone bedrock. Its impoundment had been filled completely 8 years before it failed. The failure was attributed to artesian water pressure in the bedrock developed by the seepage of water from other impoundments further up the slope, combined with tensile strains induced in the clay stratum by old underground mine workings. In a third case, a clay layer in the foundation of the dam sheared when the dam reached a height of 79 m. This caused a 240 m long section of the dam to slump. It was stabilized by the use of rock drains and toe weight.

More cases were documented of foundation problems where engineering solutions were attempted. First is the case of a dam that was founded on about 50 m depth of material overlying lava flows. There appears to have been a layer of weaker material lying over the lava flows, and when the dam had reached a height of 25-30 m, the dam slid forwards on this deep layer. The dam was stabilized by draining the layer into excavated tunnels from the existing underground mine workings, and constructing a rock buttress at the dam toe that would act as toe weight.

Another, but unsuccessful engineering solution was of the Aznalcóllar tailings dam in Spain. The dam failed in April 1998 when it had reached a height of 27 m. Length of the dam of about 600 m swung open forming a breach about 45 m wide and releasing an estimated 5.5 x 10^6 m^3 of acidic tailings that flooded over an area of approximately 2.6 x 10^3 hectares of farmland. The dam was founded on about 4 m thickness of alluvium, overlying marl, which may have contained pre-formed slip surfaces. The impoundment, about 2 km long and 1.2 km wide, was along one side of the flat valley of the river Agrio. The tailings in the impoundment ~~was~~ were particularly heavy, with a bulk density of 28 Kn/m^3. The failure occurred where a dividing dam met the main dam. In 1995, some grouting was carried out from the main dam crest to reduce leakage. 42 relief wells were placed towards the end of 1997 along the downstream toe, through the alluvium and about 1-1 ½ m into the marl, and for several months they collected 1000 m^3 /hr of seepage that was pumped back. There was some evidence to suggest that the main dam failed north of the dividing dam, prior to the bodily movement of the main dam adjacent to and south of the dividing dam on a shear plane deep in the marl.

b) Instability of the downstream slope: The dangers of allowing the phreatic surface to move so far downstream as to intersect the downstream slope of a dam are the same in tailings dams as in conventional earthfill dams. If the rate of construction raises the vertical height sufficiently slowly then construction pore pressures can dissipate, and appreciable pressures seldom develop within the dam itself creating no threat to the stability to the downstream slop. Pore pressures are raised however by the rising impoundment, which usually rises at the same rate as dam construction which means that rate of construction must be kept slow enough to allow pore pressure dissipation. Moreover, if the rate of tailings deposition is slow enough to keep a long unsaturated beach, then this can help in keeping the phreatic surface inside the dam low enough. Trouble can arise when pond level rises quickly saturating the beach and bringing the edge of the open water closer to the dam axis.

Rainwater gullies in the downstream slope can worsen the situation, and if any of them is deep enough, then the phreatic surface may intersect with it causing sloughing of the materials and creating possible circular slip surface. If this behavior continues for too long, unobserved, larger and larger rotational slips occur, endangering the stability of the whole dam.

In some cases, the phreatic surface can be moved back from the slope by the installation of horizontal bored drains. In other cases, additional drainage has been installed in the form of pumped vertical wells. While such measures might serve the purposes, in particular situations, it is advisable always to optimize the rate of raising the tailings with the rate of pore pressure dissipation, which can be measured by use of open pipe piezometers.

c) Superimposed loads: Failures had occurred when loads were added to dams retaining tailings by increasing the height of the dam while retaining the same downstream slope.

An example is cited from Lower Indian Creek lead mine, USA, 1960. The impounding earthfill dam was built in 1953 to a height of 14 m and subsequently raised several times by the addition of earthfill. In 1960, slumping occurred in the 1 on 2 downstream slope. The dam was saved by the addition of rockfill toe weight, placed with an outer slope of 1 on 3. The dam remained in service and was raised between 1971 to 1976 with cycloned sand fill, ultimately reaching a height of 25.3 m.

Another case is that of Maggie Pye china clay, UK, 1970. The 18 m high dam suffered slope failure immediately after completion of a dyke on the crest to raise the dam; following a period of heavy rain, high pore pressures, together with the added weight of the new fill were thought to be the causes for the failure. In this case, 15,000 m^3 of tailings were released.

ii. Water Management

A high water level can result from heavy precipitation events or inappropriate decanting, which ends in overtopping the embankment crest resulting in failure. In so many cases, water ponded in a tailings impoundment is removed by evaporation, pumping from a floating barge, or decanting into a tower acting as spillway that exits the impoundment through a culvert or pipe beneath the tailings dam. One of the most common causes of unintended dangerous rises of pond water levels is inadequate behavior of decants. This may be produced by debris blockage, crushing and/ or fracture of the outlet passing under the dam, or by unanticipated flood. An example of decant towers is shown in Figure (6). Damage to decant towers caused by ice has also been reported.

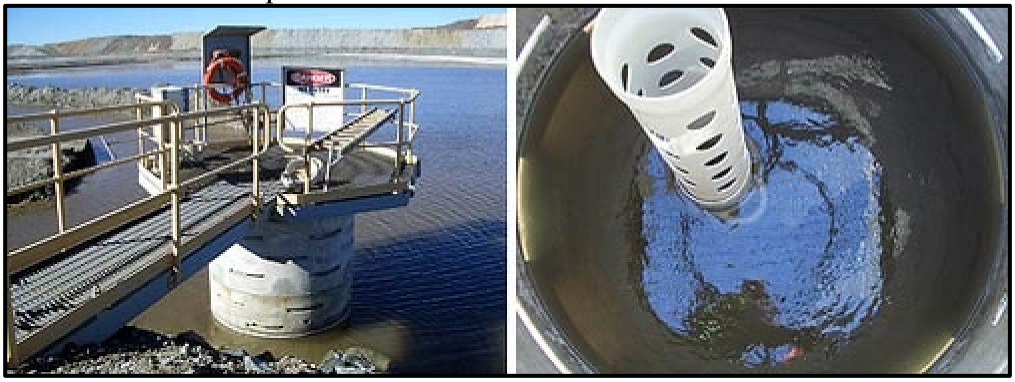

Figure 6: Decant tower with walkway (left), view inside the tower showing riser pipe and submerged decant collars (right) [15]

In Casapalca, Peru, several tailings dams were built by upstream method, up to 107 m high, which used a complex array of pipe type decant structures and inadequately sized stream bypass channels. Five separate dam failures resulted from failures in these systems. Another example is from Blackbird, USA, Cobalt mine where the metal culvert under the dam corroded and partially collapsed. Suspended tailings discharged into downstream drainage, but no embankment breach occurred. Contrary to this, the failure in 1974 of three tailings impoundments that had been built in sequence within a narrow valley for Galena silver mine, USA, when during a rain on a snow event that caused a 100 year flood, a blockage diverted a large portion of the flood into the uppermost impoundment. Its decant could not accept this large flow, causing the upper dam to fail by overtopping, leading to a cascade failure of the others. Released tailings covered about 5 acres of land, including part of a highway and main line railway.

Sufficient freeboard under all circumstances and all along a tailings dam is one of the most important prerequisites for safety. This calls for a sound water balance of the tailings disposal system considering all the components of inflow under the varying conditions of operation and the climatic conditions. Extreme situations with low frequency of reoccurrence need also to be checked. In parts of the world subjected to long periods of frost, failures have occurred as an indirect result of freezing. One example is Kimberley iron mine, British Columbia, when in 1948, slope failure occurred during a period of high snowmelt and spring runoff that raised the phreatic surface while the surface of the slope was frozen. A large tailings flows slide developed, and frozen blocks of material were seen in the flowing mass. In another reported case; extensive sloughing of downstream slope was attributed to freezing and growth of ice lenses accompanied by the development of piping during the first few days of a spring thaw.

iii. Earthquakes

Dams built by the Upstream Method are more susceptible to damage by earthquake shaking. There is a general suggestion that this method of construction should not be used in areas where there is a risk of earthquake. Dams built by the downstream method, in cases where there are sufficient volumes of the coarser fraction in the tailings, or those built from borrowed clayey fill as water retaining dams, are much less prone to damage by earthquake shaking. Harry Bolton Seed (1922-1989) had suggested in the 19th Rankine Lecture in 1979 titled "Considerations in the earthquake-resistant design of earth and rockfill dams" that it was noteworthy that no failures have been reported in dams built of clayey soils even under the strongest earthquake shaking conditions imaginable, and that all cases of slope failure reported have involved sandy soils, which strongly suggests a liquefaction type of failure.

One history case is to be cited here from the Barahona 61 m high dam, Chile, 1928. The dam was built by the Upstream Method with downstream slopes of 1 on 1. The dam failed during the Talca earthquake of magnitude 8.3, producing a breach of 460 m wide. The released tailings flowed down the valley, killing 54 people. In another case of Bellavista, Chile, 1965, a 20 m high dam built by the Upstream Method with downstream slope of 1 on 1.4, failed during the La Liqua earthquake of 7.7 magnitude. At the time, only 8 m separated the edge of the pond water from the dam crest.

Many more such failures are recorded and documented in the literature. Failure of a Tailings Dam by itself, while causing an inconvenience, may not have in some cases seriously been damaging consequences nor cause any loss of life, but this depends on the size of tailings storage, the proximity from rivers and water courses in addition to being located in remote and undeveloped areas [16] and [17]

5. Multiple Failure Modes Case Studies

While many examples of tailings dams' failures are mentioned in the preceding pages, it is worthwhile, however, to illustrate how a combination of many failure modes can result in such failure. In the following is the case of Merriespruit Harmony Gold Mine Tailing Dam in South Africa, 1944, which sheds more light on such an event.

In this case, overtopping due to an inadequate freeboard was ample to trigger static liquefaction and toe material erosion causing a massive flow slide to be initiated soon after. The Harmony tailings were quite fine-graded with more than 60% finer than 74 µm. However, these fines were also essentially cohesion-less and once an area of the dam toe was eroded, and local slopes were increased to the range of 2H:1V, static liquefaction and the massive flow slide were inevitable. To add to these factors mismanagement of operations was decisive in accelerating the failure at a time when the possibility of saving the dam was at hand.

Progress of this tailings dam had utilized a "paddock" system for tailings management. Paddock systems are relatively common in South Africa and are essentially upstream constructed tailings impoundments with little freeboard and relatively saturated dam shells [18]. For detailed information on the Paddock system, please refer to reference [19].

The mine was located near the town of Merriespruit. On the 22nd February 1994, the Merriespruit Tailings Dam failed by overtopping as a consequence of heavy rains causing static liquefaction and flow slide of part of the embankment once enough toe material was eroded away. Water mismanagement was to blame, which caused 600,000 m^3 of tailings to mobilize out of the impoundment where the flow eventually stopped 2 km away in the town of Merriespruit drowning 17 people, and scores of houses were demolished. Figure (7) shows the extent of the damage to the town and the scale of the breached embankment.

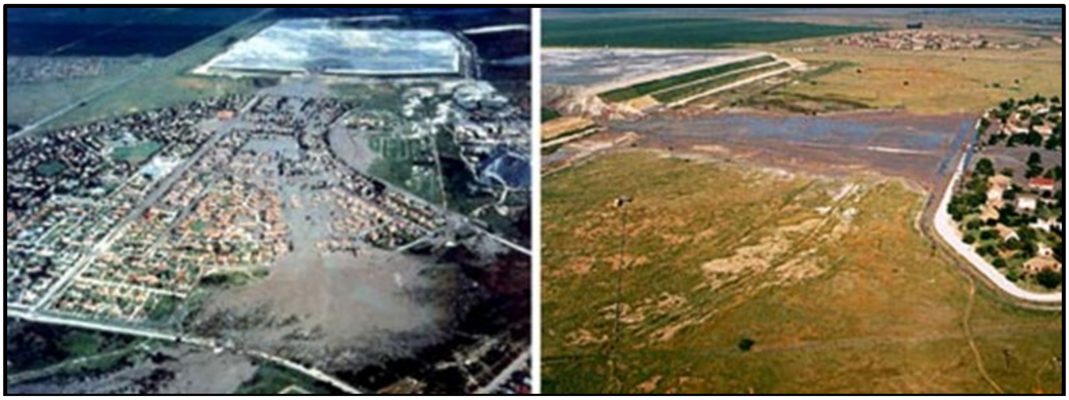

Figure7: Merriespruit tailings engulf the town.

The 31 m high embankments had problems prior to the major failure. Small slips caused the impoundment to close temporarily, and only mine water with small amounts of tailings were deposited. The deposition of these tailings caused the floating material (supernatant pond) to move to the opposite side of the impoundment away from the decant system rendering it inoperable. A satellite recorded the transition stages of the decant pond relocation as more tailings were deposited with the mine water. Heavy rains that fell on the day of the failure (30-55 mm in 30 minutes), combined with no possibility of emptying due to the idle decant system, caused the overtopping. The failure event is illustrated in Figure (8). A close up of the breach is shown in Figure (9).

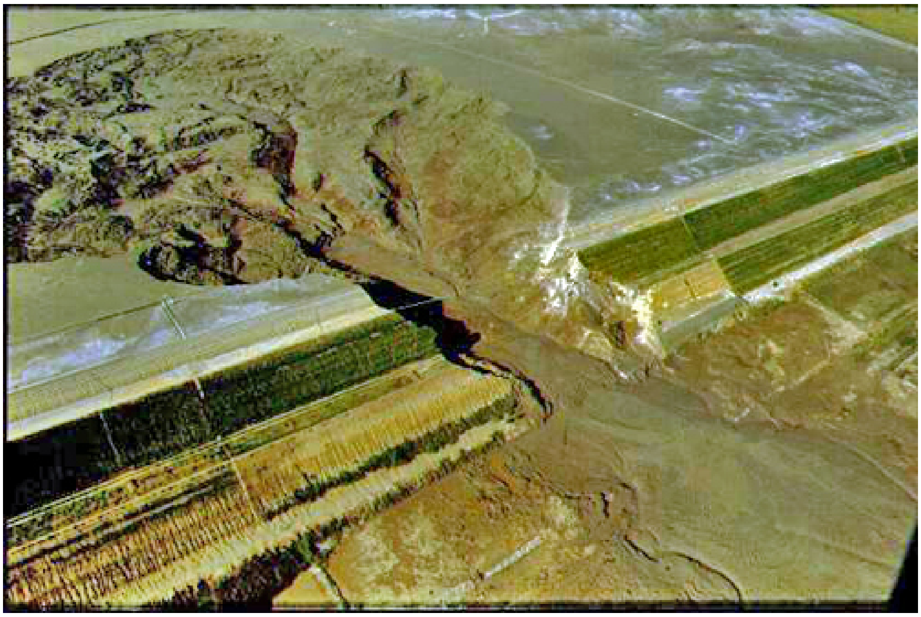

Figure 8: Aerial of the Merriespruit failure

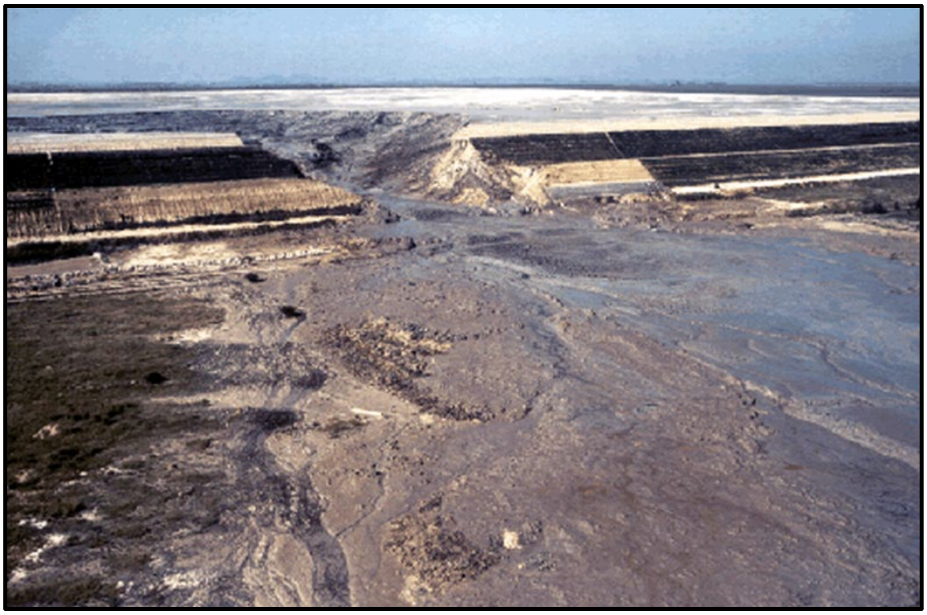

Figure 9: Close-up of the embankment breach.

The failure could have been prevented if a suitable operating manual, and emergency plan had existed and been implemented successfully. The operation of the facility leading up to the major failure was outside the designed operating procedures. The facility was closed as a result of earlier embankment slips but still process water and tailings were being discharged to the impoundment. This suggests that an emergency plan was not in place to implement intervention actions, or it could have been that documented procedures were overridden by an individual making an executive decision. Whichever the case may have been, the lack of understanding of the operational procedures and the seriousness of the events prior to the main failure had not been realized. Moreover, if a well-structured and executed operational plan had existed, then the tailings operator(s) would have known to intervene, continually monitor the closed impoundment for change and prevent further discharge to the facility.

The position of the supernatant pond and its ability to decant was lost which suggests that the monitoring procedures were inadequate to notice the change in the pond geometry and location. Alternatively, the tailings operators may not have been trained to realize the consequences of pond migration away from the decant system as a result of single point deposition of the process water and tailings. A management structure documenting individual responsibilities, operating procedures and contingency measures would have been sufficient to identify and prevent the initial localized slips and how the facility was to be managed thereafter. Having a suitable tailings management system implemented could have prevented this failure from occurring [20].

The importance of rigorous management tailings dam operation cannot be over emphasized, such plans were prepared and followed in cases such as the

Endako Mine (British Columbia, Canada and AngloGold, South African) [21].

6. Conclusions, Lessons Learned and Recommendations

Many conclusions can be drawn from a thorough study of documented tailings dams' failures and incidents. Lessons can be drawn from these studies which enhance the current knowledge of dams' behavior and faults committed in their design, construction and operation. The consequences of these failures have been heavy economic losses, environmental degradation and, in many cases, human losses. Even with the current improved technology for the design and the improved construction and operation of Tailings Dams, failure of such dams hascontinued to occur, which should be enough reason to continue research and improvements to make such dams as risk free as possible.

The conclusions and lessons learned from these failures shall be considered as additional resources for having safer tailings. Such conclusions and lessons, may be summarized by, but not limited to, the following:

6.1 Data related to causes and types of reported tailings failures, and incidents should be looked at with great caution; inaccuracies could have crept into them since different people had reported them, using different forensics, at different times, and may be with different purposes. Information on failures of tailings dams due to incorrect management, which have resulted in extensive long lasting and costly damages to the environment and considerable human fatalities, may be the most misleading for the obvious reasons of covering up attempts to relieve the operators of their personal responsibility and the heavy penalties that they may face. 6.2 Available statistics indicate that total number of accidents, using an estimate of 3500 active tailings dams in the world, has gone down from over 50 during the 1960s-1980s to less than 30 in the 2000s - 2010s. This could be due to more awareness of the following:

- The upstream construction method is a hazardous one and especially so in seismic areas. Indeed, Chile has forbidden such a construction type.
- Failures at operating facilities are more than five times more than at closed abandoned facilities. Which can mean that stricter control over operations activities should be exercised.
- The percentage of all failures and incidents cases whose causes are reported as "unknown or not available" makes 26% of all cases. Subject to this limitation, it may be seen from the remaining cases that:
  - (i) Overtopping, seismic and slope stabilities are the strongest categories making 42 to 48 accidents for each, whereas;
  - (ii) Seepage, structural inadequacy and foundation come in a second category making 20-25 accidents for each, and;
  - (iii) Erosion and external erosion are the smaller category with 11 accidents.

6.3 The consequences of failures are not necessarily correlated, in one way or another, with dam height or pond volume, for example:

- (i) A small (300,000m^3) release at Stava tailings dam failure (Italy, 1985) killed hundreds of people. That was because the valley downstream funneled the release toward a village a few kilometers away. Stava became one of the most mortiferous tailings accidents in history, whereas,
- (ii) The more recent Samarco (Brazil, 2015) huge release ran hundreds of kilometers, killing dozens of people only, and;
- (iii) Mount Polley (Canada, 2014) mostly ended in a lake nearby, with no victims [22].

6.4 The causes in many failures could be attributed to lack of attention to details. Original design heights are often exceeded, and the properties of the tailings can change with time. Lack of water balance may be due to rising phreatic levels causing local failures that produce crest settlements, which can lead to "overtopping". Other causes include problems of foundations with insufficient investigations, inadequate or failed decants, slope instability, erosion control, structural inadequacies and additional loading of impoundments.

6.5 In the design, construction, operation and closure of dams, impoundment risks potential to downstream shall be minimized by emphasis on the following requirements:

- (i) Detailed site investigation by experienced geologists and geotechnical engineers to determine possible potential for failure, with insitue and laboratory testing to determine the properties of the foundation materials.
- (ii) Application of state of the art procedures for design.
- (iii) Expert construction supervision and inspection.
- (iv) Laboratory testing for "as built" conditions.
- (v) Routine monitoring.
- (vi) Safety evaluation for observed conditions, including "as built" geometry, materials and shearing resistance. Observations and effects of piezometric conditions.
- (vii) Dam breaks studies.
- (viii) Periodic safety audits.

6.6 Regulatory Authorities should be more concerned about the safety of Tailings Dams, which come under their jurisdiction and should require periodic reviews carried out by appointed specialists. In some countries, approval had to be obtained for specific stages of construction, causing the stability, general condition and safety to be automatically checked from time to time [23].

6.7 Monitoring of dams is very important to detect any progressive conditions leading to failure before it occurred. Technologies and modern instrumentation exist today, which together with visual inspections can detect the main factors directly influencing dam safety, these are:

(i) Seepage discharge through the dam itself, through the foundations and abutments.
(ii) Position of the phreatic surface and any danger that it might emerge on the slope
(iii) Pore pressure development in the embankment
(iv) Seismicity and induced dynamic pore pressure
(v) Horizontal and vertical movement of the starter dam crest and of the downstream slope
(vi) Amount that the dam crest is above water level (Freeboard)
(vii) Beach width, which should be as large as possible
(viii) All tailings placement procedures [24].

6.9 In a majority of failures associated with large tailings damages, the absence was noticed of a risk assessments study with a meaningful updated emergency action plan. The same importance attached to these for conventional dams must also be given to tailings dams. They are valuable assets, which can help in minimizing the damages and human fatalities through fast actions for evacuation of endangered population and for quick remediation measures. This requires good communication arrangements with the public to raise awareness and secure cooperation in an emergency case. Effective communication between site management and headquarters helps to take prompt relief and repair actions, such communication allows the community as well as the responsible organizations to participate in planning the best emergency preparedness arrangements.

If the community understands the real consequences of a spill before one occurs, it will be better prepared to deal with the emergency. Among other things, the general anxiety level will be reduced. If, on the other hand, the community is uninformed, its anxiety and mistrust may prevent the mining company from being able to communicate with it effectively during such an emergency [25].

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Dam Safety: Sediments and Debris Problems

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Abstract

Sedimentation of reservoirs has its negative impacts on dams, first by reducing useful storage, altering the benefit/cost ratio originally calculated for the dam, and second by reducing the dams' capacity for flood routing; increasing flooding hazards on the dam itself and for the downstream. More problems can be created by sediments and floating debris during floods on outlet structures by clogging them and thus creating dangerous situations, or damage trash screens leading to even more problems. If these debris and coarse sediments are allowed in, then they may damage dam structures such as gates, spillways intakes in addition to chutes, stilling basins and power penstocks by the mechanical abrasion impacts of such sediments on them. Frequent inspections, especially after floods must be made to ensure proper functioning of such structure and take actions for reducing the damage. In small reservoirs, dredging; although it adds to maintenance cost, may ease the problem, but in very large reservoirs, this may prove unpractical. Designers, therefore, have a duty to consider sedimentation problem seriously in the initial stages of design by: checking the anticipated accumulation of sediments, allowing enough storage free from siltation, foreseeing their negative impacts on intakes and outlet structures and taking design measures to reduce these impacts. At the same time, dam stability calculations shall have to provision for the anticipated new conditions of silting up at the face of the dam. Operators of dams, on the other hand, shall have to keep open eyes for all the negative issues created by sediments and floating debris, repairing damages caused by them and take measures to reduce their impacts in the future.

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1. Introduction

Storage reservoirs have a very important role to play in water resources management all over the world. By storing excess water in flood season, this water is put to the best use possible for irrigation and power generation. By this manipulation, reservoirs modify the shortcomings of the hydrological cycle and modulate river flows in a way to maximize the anticipated benefits and reduce negative impacts. The continuously increasing demand for water as a consequence of world's population growth and expanding industry drives towards increasing storage capacity by constructing more dams. It is pointed out that whereas the future demand will require additions, the present capacity is being continually eroded by siltation. It is estimated that on a worldwide basis ,the replacement cost of the capacity annually lost to siltation is around $6 billion.

The United States Bureau of Reclamation (USBR) has recognized the magnitude of the problem confronting the project planner in estimating the rate of deposition, and the period of time before the sediments will interfere with the useful function of the reservoir. So, the Bureau has recommended that provisions should be made for sufficient sediment storage in the reservoir at the time of design so as not to impair the reservoir functions during the useful life of the project or at least during the period of economic analysis. The replacement cost of storage lost due to sediment accumulation in USA's reservoirs amounts to millions of dollars annually [1]. Review of the world picture of erosion and sediments' yield from drainage basins shows that the world average yield at the ocean level is a modest 500 ppm, but large variations exist, and local values can be much higher due to natural conditions, human interference and natural events that further affect sediment yields [2].

Most river systems around the world actually show decreased sediment loads, because of trapping by upstream dams. Estimates of sediment that reached the ocean, or at least deltas, under pre-human-disturbance conditions have been in the range of roughly 15–20 billion tons per year. An estimate has shown that at catchment-level human disturbances have increased the erosion of sediments from uplands and its delivery to rivers by about 2.3 billion tons per year, but that the net effect has been a reduction in sediments' loads of rivers by an estimated 1.4 billion tons due to sediments trapping in reservoirs. An extrapolated estimate from 633 large reservoirs to over 44,000 smaller reservoirs had concluded that more than 53% of the global sediment flux in regulated basins is potentially trapped in reservoirs, or 28% of all river basins, for a total trapping of 4–5 billion tons per year [3].

Depletion of reservoir storage by sediment deposits is a problem which reduces dams' benefits with the passage of time and influences the project economics as the available volume of usable water decreases. Moreover, safe operation of any dam in case of floods can be impaired by the accumulation of sediments in the reservoir which alters the (Elevation vs. Capacity Curve) from the original one initially adopted for design rendering erroneous results and may give the operators false sense of security in flood operations.

More problems related to sediments accumulation close to intakes result in increased through flow with more and more coarser particles entry. Dam structures such as gates and spillways, in addition to power penstocks can be damaged by the mechanical abrasion impacts of such sediments. Floating debris in more frequent overflows causes similar damages in these structures; unless an eye in kept for the continuous cleaning and removal of the accumulated debris. Sedimentation process and silt accumulation in front of dams may even reduce factors of safety for the originally calculated dam stability which can be altered due to changes of reservoir morphology at the close vicinity of the dam.

2. Dam Safety Issues Related to Loss of Storage and Operational Routines

The siltation is a rather minor problem for many dams with large reservoirs but may reduce within decades the possible long life of 50 % of them, and may be a key problem within few years or few decades for over 10% of large or small dams. The total reservoirs storage in the world is about 7000 km3 for 50 000 large dams; this includes very roughly:

- 5000 km3 for 10 000 hydropower dam,
- 500 km3 for 30 000 irrigation or water storage dams, and
- 1500 km3 for dams devoted to multipurpose uses, including hydropower generation.

Annual total rivers inflow worldwide is about 40 000 km3. The annual sediment storage may presently reach 0,6% of the total available1500 km3 storage capacity of multipurpose reservoirs but the large part, however, is in the designed dead storage of hydropower dams [4].

In flood control and irrigation dams, designers may encroach on the dead storage level in favor of increasing the active storage to improve the overall economicindices of these dams since the benefits are quite proportional to the active storage, but this assumes, however, including design and operational measures to routinely sluice as much as possible of the accumulated sediments by providing bypass tunnels, dredging facilities or formulating future dam upgrading plans.

Accumulated sediments removal by suction using hydraulic pumps on barges with intakes may work for some dams in special cases. If cohesive sediments have “set up,” cutter heads may be required to break up the cohesive sediments. Dredging is expensive, so it is most often used to remove sediment from specific areas near dam intakes. If there is a sufficient hydrostatic head over the dam, it can create suction at the upstream end of the discharge pipe to remove sediment and carry it over the dam as a siphon. This hydro-suction is typically limited to reservoirs less than 3 kilometers in length, and to low elevations, where the greater atmospheric pressure facilitates the function of the siphon. In China, hydraulic suction machinery is commonly used, but sediments within the reservoir are stirred first with hydraulic and mechanical power, then to discharge the highly concentrated sediment-laden water out of the reservoir through siphons by the help of water head difference between upstream and downstream of the dam.

If a reservoir is completely drawn down, then mechanical removal can be employed using scrapers, dump trucks, and other heavy equipment to remove accumulated sediments. While still costly, mechanical removal is commonly less expensive than hydraulic dredging, and can remove coarser sediments, but it requires the reservoir to be drawn down far enough to expose coarse sediment. Mechanical removal is best adapted to reservoirs that remain dry for parts of the year such as flood control reservoirs. Cogswell Reservoir on the San Gabriel River, California, was mechanically dredged in 1994–1996, with 2.4 million m3 removed and taken to a nearby upland disposal site, at a cost of $5.6/m3. Another 2.55 million m3 has been identified as requiring excavation following a 2009 wildfire that increased erosion in the catchment [5].

Another case where control of the sediments' problems may dictate imposing certain risk reduction plans, and continuous maintenance works are found in the case of Devil's Gate Dam and Reservoir, which serves as a good example [6]. Devil's Gate Dam is a 100 foot (30.5 m) high concrete gravity arch dam located in the City of Pasadena approximately 1.5 miles south of the base of the San Gabriel Mountains. Construction of the dam began in May 1919 and was completed in June 1920 for flood control and water conservation. The original capacity of the reservoir was 7,423,000 cubic yards (5.66 million m3) below the original spillway elevation of 1,054 ft. This dam represents a critical component of the Los Angeles County Flood Control District's infrastructure, providing flood protection to the cities of Pasadena, South Pasadena, and Los Angeles, as well as the 110 Freeway, and numerous recreational facilities along the Arroyo Seco, including the Rose Bowl and Brookside Park. Sediments and debris are captured in the reservoir during storm events, and sediment removal efforts have previously taken place at the reservoir in order to ensure correct functioning of the outlet works and/or to maintain reservoir capacity.

Two major Devil's Gate Reservoir sediment removal projects were undertaken in 1994, when 190,000 cubic yard (145,000 m3) of sediment were removed. Sediment was trucked off site via a maintenance road just west of the dam which exits on to Oak Grove Drive. Two smaller sediment removal operations also took place with 14,000 cubic yard (10,700 m3) removed in 2006 and 3,800 cubic yard (2900 m3) removed in 2009.

Following the devastating 2009 Station Fires in California, denudation of over 160,000 acres (64,800 ha) of forests had left vast areas of the catchment in San Gabriel Mountains denuded and susceptible to sediment erosion. During the 2009 – 2010 storm season 1,000,000 cubic yard (765,000m3) of sediments and debris were deposited into Devil's Gate Reservoir. For comparison; that one-year volume was ten times the total amount deposited in the reservoir during the previous sixteen years. The sudden sediment accumulation buried one of the dam's outlet gates and threatened to block the other outlets and spillway ports. Additionally, sediments reduced the reservoir's volume, and it no longer had a capacity to safely contain another major flood and debris event. Figure (1) shows Devi's Gate Reservoir Sediment History Profile after 2009 Station Fires storms [7]. In Figure (2), the diagram shows past accumulation of sediment and the remaining free capacity in 2011cross section of the dam showing sluice gate, outlet valve and sediments accumulation in 2011 [7].

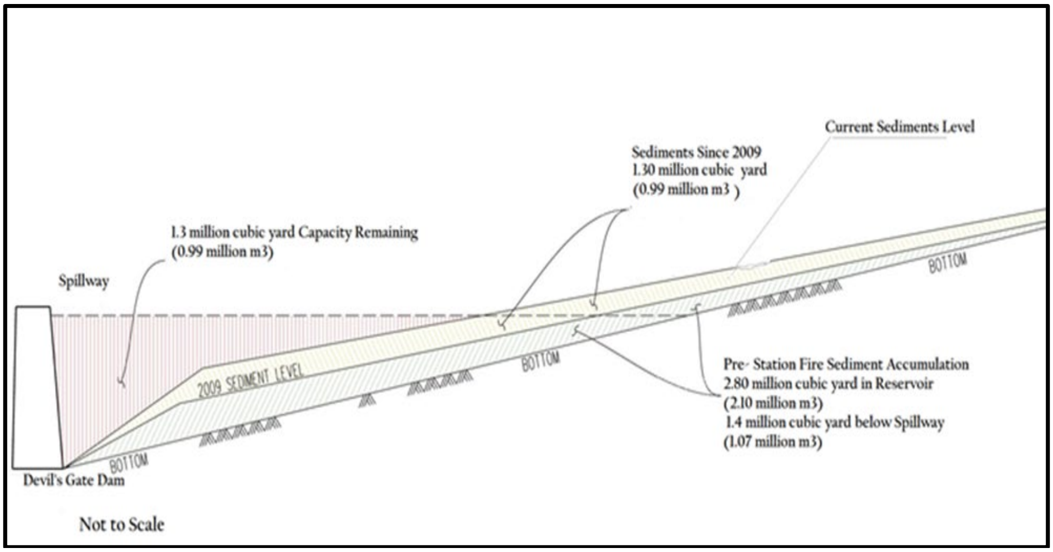

Figure 1: Devi's Gate Reservoir Sediment History Profile after 2009 Station -Fires storms [7].

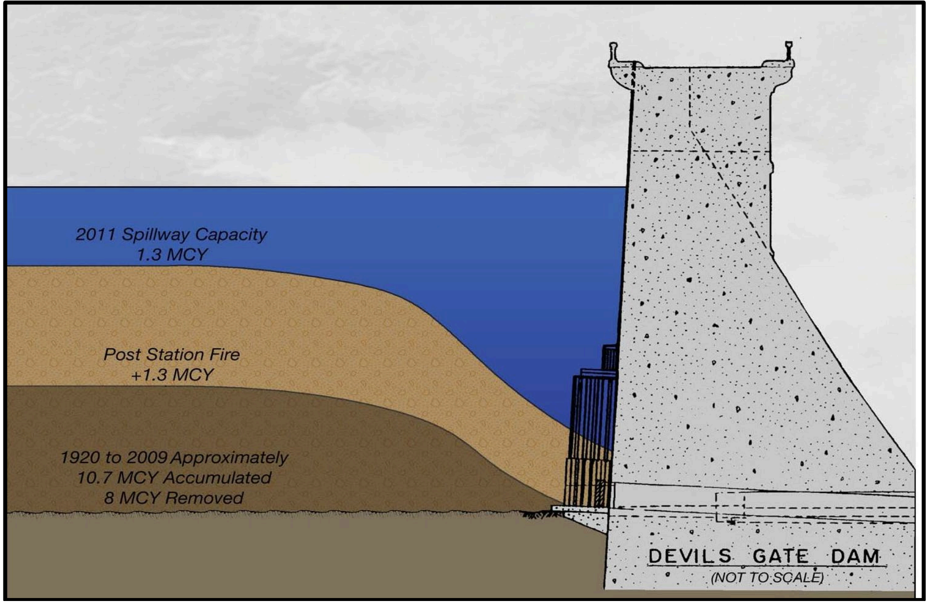

Figure 2: Devil's Gate Dam Cross Section showing accumulated sediments [7].

In a temporary measure to minimize the anticipated impacts until the sediment would be removed, a Risk Reduction Plan was developed that included an interim operating plan, physical dam modifications, and a Flood Hazard Warning and Contingency Plan. These measures were estimated to cost a $960,000 capital outlay as well as an annual maintenance expenditure of up to $1,250,000 depending on the severity of each of following storm seasons.

The Interim Measures Project included minor dam modifications to help keep debris from plugging the outlet works and allow for removal of up to 25,000 cubic yard (19,000 m3) of sediment per year from the dam face until the project associated with a proposed Environmental Impact Report (EIR) and plan would be started. In 2011, 13,000 cubic yard (9940 m3) were removed from the dam face. In 2012, approximately 1,525 cubic yard (1166 m3) of sediment and 419 cubic yard (320 m3) of green waste were removed from the dam face and hauled to Johnson Field and Scholl Canyon Landfill, respectively. In 2013, 1,200 cubic yard (900 m3) of sediment and 12 to 14 loads of green waste were removed from the dam face and hauled to same landfill. These interim risk reduction measures were to be implemented until the sediment would be removed from the reservoir following the completion and approval of the Environmental Impact Report (EIR). Photographs showing sediments removal operation during 2009 – 2011 are presented in Figures (3,4, and 5)

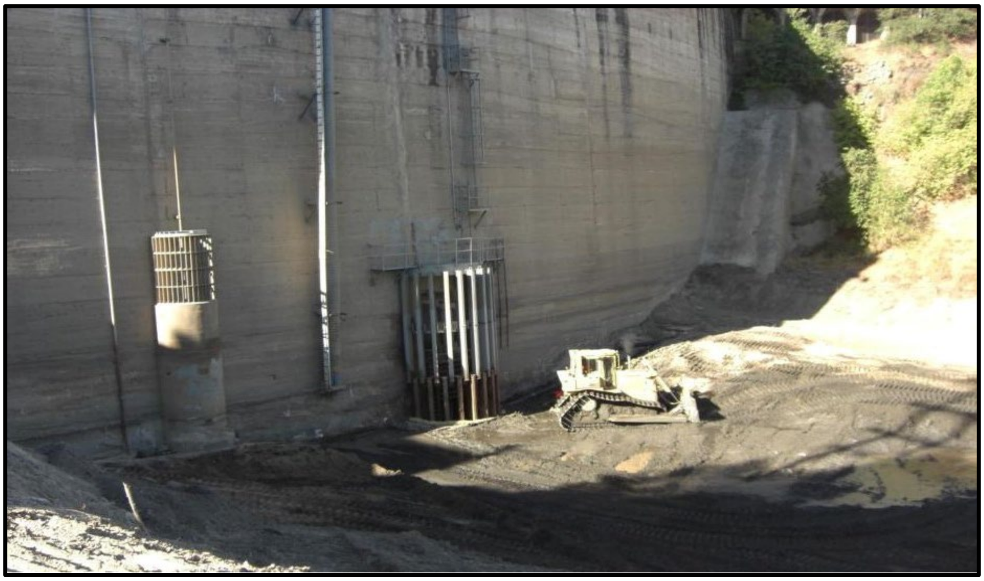

Figure 3: Intake Structure cleared pre- Station Fires of 2009 (Photograph dated on 8th June 2009 October 2010) [6].

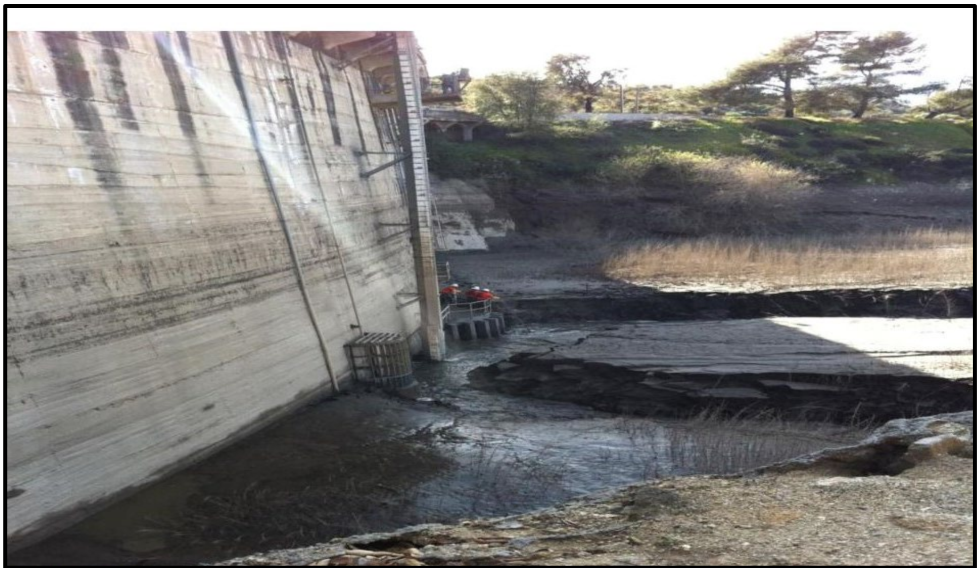

Figure 4: Sediments Threatening Sluice Gate Post- Station Fires of 2009 (Photograph dated on 3rd October 2010) [6].

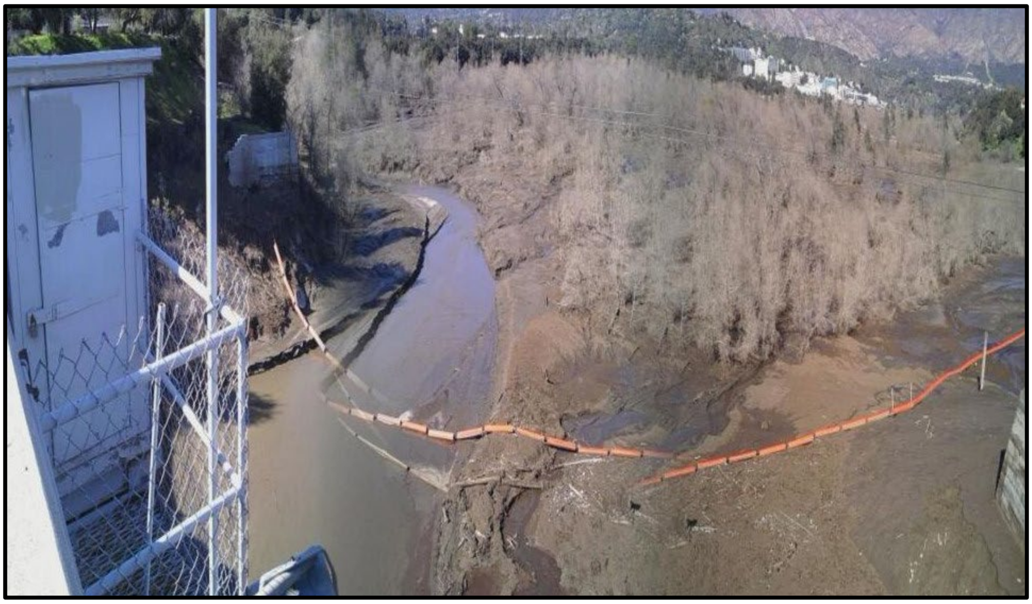

Figure 5: Upstream view of the Reservoir upstream of the dam (Photograph is dated 2nd February 2010) [6].

Under the mentioned EIR a reservoir storage design capacity of two Design Debris Events (DDEs) below the dam's lowest spillway was determined to be the standard acceptable level of risk at Devil's Gate Dam and Reservoir. The DDE capacity volume is determined using the January 2006 County of Los Angeles Department of Public Works Hydrology Manual and the March 2006 County of Los

Angeles Department of Public Works Sedimentation Manual. The reservoir, therefore, was likely to have sufficient capacity to experience a design level storm, or several smaller but significant debris events, and still maintain capacity of at least one Design Debris Events during the lengthy environmental and construction processes to remove the debris. Further, it should be noted that additional criteria in special circumstances related to dam safety may also dictate the need to remove sediment from a reservoir, which includes:

i. Depending on the structural stability of the dam, the height of sediment against the dam may need to be limited, as sediment weighs more than water and increases the forces on the dam during an earthquake.
ii. The volume of sediment accumulation may also be limited to preventing sediment from blocking valves operations. If the debris blocks the outlet valves, then they cannot be used to regulate storm flows or to empty the dam during an emergency.

The Proposed Project for Devil's Gate Reservoir was to remove the sediments to restore the design capacity; volume for two DDEs below the spillway elevation of 1,040.5 feet, and establish a reservoir management system to maintain the flood control capacity of the reservoir. This included removal of approximately 2.9 million cubic yard (2.2 million m3) of existing excess sediment in the reservoir in addition to any additional sediment received during the project sediment removal phase. Primary project objectives included:

i. Reducing flood risk to the communities downstream of the reservoir adjacent to the Arroyo Seco by restoring reservoir capacity for flood control and future sediment inflow events.
ii. Supporting sustainability by establishing a reservoir configuration more suitable for routine maintenance activities, including reservoir management,
iii. Removing sediment in front of the dam to facilitate an operational reservoir pool to reduce the possibility of plugging the outlet works with sediment or debris during subsequent storm events,
iv. Removing sediment placed at Johnson Field during the Devil's Gate Reservoir Interim Measures Project,
v. Supporting dam safety by removing sediment accumulated in the reservoir in a timely manner to ensure the ability to empty the reservoir in the event of a dam safety concern; and,
vi. Delivering the sediment to placement or reuse facilities that are already prepared and designated to accept such material without native vegetation and habitat removal [8].

In a resent update released by Los Angeles County Public Works on their website on 4th February 2020, it described the Devil's Gate Reservoir Restoration Project as a four-year continuing effort to increase flood protection for communities downstream of Devil's Gate Dam and restores habitat within a popular section of the Arroyo Seco Watershed. Los Angeles County Public Works will still remove the remaining 1.7 million cubic yards (1.3 million m3) of sediment from the reservoir immediately behind the nearly 100-year-old dam. In addition to providing flood relief to communities that have endured nearly a decade of elevated flood risk along the Arroyo Seco, the project will establish a permanent stormwater maintenance area that allows for the creation of 70 acres (28.3 ha) of enhanced habitat and recreational opportunities for local communities; updates on the status of the work began since 10th September 2018 [9].

One more case history published in 2016, which explains safety hazards facing multipurpose dams, as contrasted to the single use of flood control in Devi's Gate Dam, is that which has resulted from siltation in Jor Dam reservoir in Malaysia. Jor reservoir was constructed to assure flow regulation for the downstream hydroelectric power station, as well as to provide storage for flood control. The reservoir is confined by two earthfill dams namely Jor Dam and Jor Saddle Dam, which form part of the Batang- Padang hydroelectric system. The power station has a rated head of 406.8 m and has a total installed capacity of 150 MW. The reservoir covers a total area of 0.32 km2 and has an original total storage volume of 3.17 million m3 with approximately 2.33 million m3 as live storage for hydropower power generation, and 0.84 million m3 as dead storage for sediment accumulations below the minimum operating water level of 480.06 m. The Batang-Padang scheme was completed in 1968, and since commencing no detailed study was conducted to determine the sedimentation rate and storage capacity change. In order to estimate the reservoir sedimentation at Jor reservoir, two bathymetric surveys up to three meters above the full supply level of the reservoir and combined with topographic surveys to a boundary of twenty meters from the reservoir banks were conducted. The two successive reservoir surveys conducted in 2007, and 2010 were compared to estimate the reservoir sedimentation and storage loss. It was estimated that in the year 2007, the gross storage had been reduced to 2.12 million m3 (33% reduction) when live storage stood at 1.95 million m3 (17% reduction), and the dead storage at 0.18 million m3 (79% reduction). While the 2010 surveys showed that both the live and dead storage were reduced further from year 2007 and the residual live and dead storages' volumes at 2010 were about 1.77 million m3 (24% reduction) and 0.05 million m3 (94% reduction), respectively. Overall, the total storage of the reservoir surveyed in 2010 has further decreased by 43% from the original one, and the remaining percentage is only 57%.

Table (1) shows the storage allocations between the two successive surveys, while the (Elevation vs. Storage curves) in Figure (6) show an upward movement of these curves from the original and between the years surveyed, which indicate a continued loss in storage. It is estimated that the annual sedimentation rate of 3.3% is an alarming figure which is greater than the world average of 1% [10].

Table 1: Storage allocation for Jor Reservoir Storage

Storage Allocation	Original (1968) (m^3)	2007 Survey (m^3)	2007 Storage (%)	2010 Survey (m^3)	2010 Storage (%)
Dead Storage	840,000	178,418	21	49,941	6
Live Storage	2,330,000	1,945,306	83	1,769,971	76
Total Storage	3,170,000	2,123,724	67	1,819,912	57

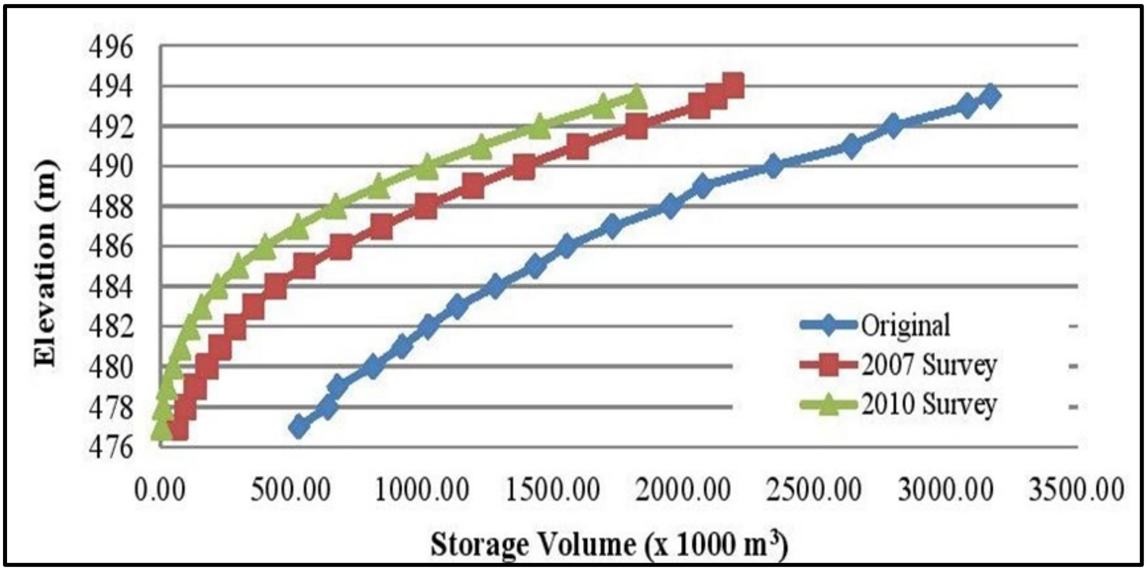

Figure 6: Elevation Storage Curve for Jor Reservoir Showing Continued Loss of Storage.

The curves in Figure (6) show that Jor reservoir has lost almost half its storage capacity due to sedimentation between 1968 and 2010. The data analyzed in the study was used to develop a revised Elevation-Storage Curve which is crucial for dam operators to plan for the future power generation and flood control.

The revised data was also expected to be used for future dam safety study and flood studies.

Since the sediments are deposited vastly throughout the reservoir, the data also enables the engineers to accurately determine the storage volume available at various elevations. Sedimentation in such a reservoir contributes to serious hazard to the overall safety of similar dams, especially in area of mountainous terrain. Periodic reservoir surveys have shown that the deposited sediment volume has increased many folds since construction contributed mainly by the uncontrolled activities upstream. The operators of the dam have been working tirelessly to address the issue by carrying out continued reservoir surveys and immediate mitigation measures such as dredging and plant life extension works [10].

It was demonstrated in the two previous examples that sedimentation of small and medium sized reservoirs can have dramatic impacts on the operation of their associated dams causing a reduction in benefits and may cause safety risks to downstream areas. This, however, may not hold to be the case for dams with sizable and very large reservoirs. Such sedimentation impacts may not impair safety and do not pose immediate risks, but in the long run certain measures may still need to be taken to enhance operational procedures. To give an example on this, the case of Mosul Dam in Iraq is cited here.

Mosul Dam is one of the such dams with a very large capacity of 13.14× 109 m3, which include a provision of 2.03 × 109 m3 for routing of the 1:1000 year flood even with two of the spillway gates not functioning and with the reservoir reaching elevation 335.00 (m.a.s.l) [11]. While the live storage provides water for irrigation and power generation and can still be used to route floods of an average year when at the same time filling the reservoir to the maximum operation water level of 330.00 (m.a.s.l. Measurements and sediment load calculations of the Tigris River at Mosul city, 40 km downstream of the proposed dam site prior to its construction had shown that the mean suspended sediment load amounted to 44.5 million tons per year, and by assuming the bed load was approximately 10% of the suspended load, the total sediment load came to 49 million tons per year. It was further assumed that the unit load of the submerged sediments was about 1100 kg/ m3; so the total sediment volume was reckoned to be 45 million m3/year which corresponds to a sediment yield of 886 m3/year/ km2 of the catchment area [11]. The design levels were selected for the 113 m high earthfill dam based on the Stage-Capacity Curve of the reservoir originally developed by the Finish Consultants Imtran Voima (IVO) in 1968, which is shown in Figure (7). These levels, including bottom outlets intake level and power intakes level with corresponding storages, are shown in the following Table (2) [12]. For the design assumed economical life of the dam of 100 years the total volume of sediments in the reservoir will amount to 4.5 × 109 tons.

Based on the assumption that the deposited sediments are uniformly spread within the reservoir, then these sediments will just reach the bottom sill of the bottom outlet inlet structure which means that these bottom outlets, which are intended for the release of water supply during the dry seasons while the power station is shut down, will not be used theoretically to flush any sediments during the lifetime of the project.

Regarding flood management, the reservoir still has ample storage to use in such events. The top level of the dead storage was also selected to be at elevation 300.00 (m.a.s.l) which corresponds to the minimum operation water level. This elevation still leaves a safe cushion of water of 17.0 m above the power intakes level which is located at elevation 283.00 (m.a.s.l)

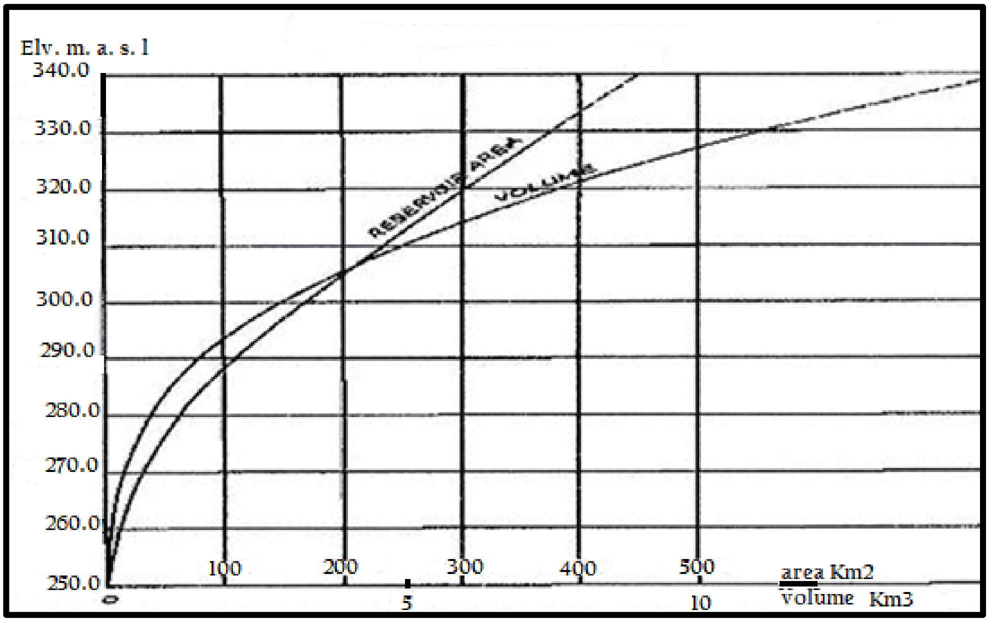

Figure 7: Stage- Volume- Area Curves for Modul
Dam Reservoir Based on Imtran Voima (IVO) survey .

As it can be concluded the dead storage level of 300 (m.a.s.l) theoretically will give enough safety for the dam against sedimentation, provided the assumption of uniform sedimentation in the reservoir holds to be true; but this cannot be guaranteed.

Table 2: Important Design levels of and Storages at Mosul dam.

	Level m (a. s. l.)	**Volume X 10^9 m^3**
Bottom outlets Intake	270	
Power Intakes	283	
Dead Storage Level	300	2.95
Sediment accumulation in 100 years	270	4.5
Maximum Operation Water Level	330	11.11
Active Storage		8.16
Flood Level in 1: 1000 year flood and two of the spillway gates not functioning	335	

One study based on a bathymetric survey performed in 2011gave the sediments configuration in the 25 years of dam operation from in 1986 till 2011 [13], and the survey materialized in constructing new (Stage-Volume) and new (Stage- Area Curves) as shown in Figure (8).

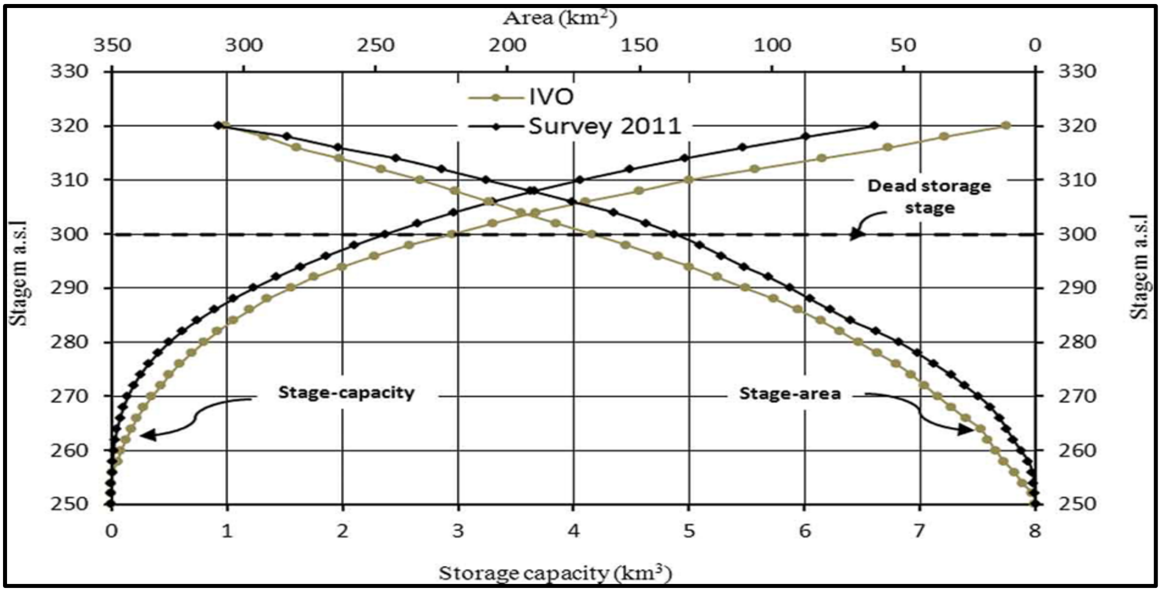

Figure 8: Stage-Volume and Stage- Area Curves of Mosul Dam.

The bathymetric survey was carried out when the reservoir water level was at elevation 320 m (a.s.l.), thus the plot indicates the changes up to this level. The storage capacity of the reservoir at pool elevation of 320 m (a.s.l.) was shown to be 7.749 ×109 m3 for the original operation curve and 6.606×109 m3 for the 2011 survey, giving a difference in storage capacities of 1.143×109 m3, which were lost to sedimentation. This difference represents the total storage loss due to sediment deposition throughout the operational period, and represents 14.73% of total storage. This implies that the annual sedimentation rate is 0.59%, which is less than the worldwide rate of 1% and that of the Middle East of 1.02%. In view of the above, the annual sedimentation rate for Mosul Reservoir during 1986–2011 was calculated as 45.72 × 106 m3/ year, (which is very close to the figure of 45.0×106 m3/year used for the design as mentioned already).The study has suggested that the reservoir will silt up completely up to the dead storage level at elevation 300.00 (m.a.s.l) after 169 years, but it neglected the fact that after 100 years, sedimentation would prevent proper operation of the bottom outlets at elevation 270 m (a.s.l.), and power generation with the power intakes being at elevation 283 m (a.s.l.).

Furthermore, using Figure (8), the live storage capacity of the reservoir was derived as 4.797×109 m3 at elevation 320 m (a.s.l.) in 1986 and 4.234 × 109 m3 for the same level in 2011. These results show that the sediment deposited in 25 years within the live storage up to this level was 0.5657 × 109 m3, which represents 49.5% of the total sediment deposited within the reservoir. This implies that the live storage capacity was reduced by 11.8% over the period 1986 – 2011.

The bathymetric survey showed that most of the sediment was deposited within the upper zone of the reservoir where the River Tigris enters the reservoir, and the amount decreases towards the dam site.

Although the results of this bathymetric survey and the prediction that the reservoir at Mosul Dam Reservoir will be silted up far beyond its economic life, it neglected, as mentioned already, the fact that after 100 years, sedimentation would prevent proper operation of the bottom outlets and stops power generation. Moreover, there are the uncertainties that must be considered with respect to the distribution of sediments during this long period of time which may still interfere with its operation. As the coarser sediments settle in the head of the reservoir forming delta, a large part of the fine sediments transported in suspension or as wash loads are carried beyond the delta after which they settle out to form the bottom set bed. They are more evenly spread than coarse sediment until delta accumulation slides or collapse approaching the dam itself. Therefore, distribution of sediments is highly uncertain dependent on reservoir circulation and stratification. Then the turbidity currents which is are the other important transport modes for fine sediments are formed when the turbid river inflow plunges below the clear reservoir water and continues as a density underflow which can also generate slides and slumping. In Figure (9), an illustration of this mechanism is illustrated, and it follows; therefore, that bathymetric surveys may be needed frequently in the future to ensure that the accumulation of sediments against the upstream surface of the dam will not reach the level of the bottom outlets intakes [14] and [15]. The reservoir survey intervals normally are based on individual site characteristics. At reservoirs losing capacity very slowly, a survey interval on the order of 20 years or even longer may be adequate [15].

It is important to mention here, that the operation of Mosul Dam had undergone a drastic change since 2006, which came out of security precaution since the dam was and, still is, suffering from foundation deterioration due to dissolution of gypsum, which is pausing in grave safety risks. The decision was taken then to lower the maximum storage water level from 330 m (a.s.l.) to 319 m (a.s.l.) which has rendered an extra volume for flood routing if the water level exceeded temporarily this level, but it is not known so far how this will affect the siltation of the reservoir.

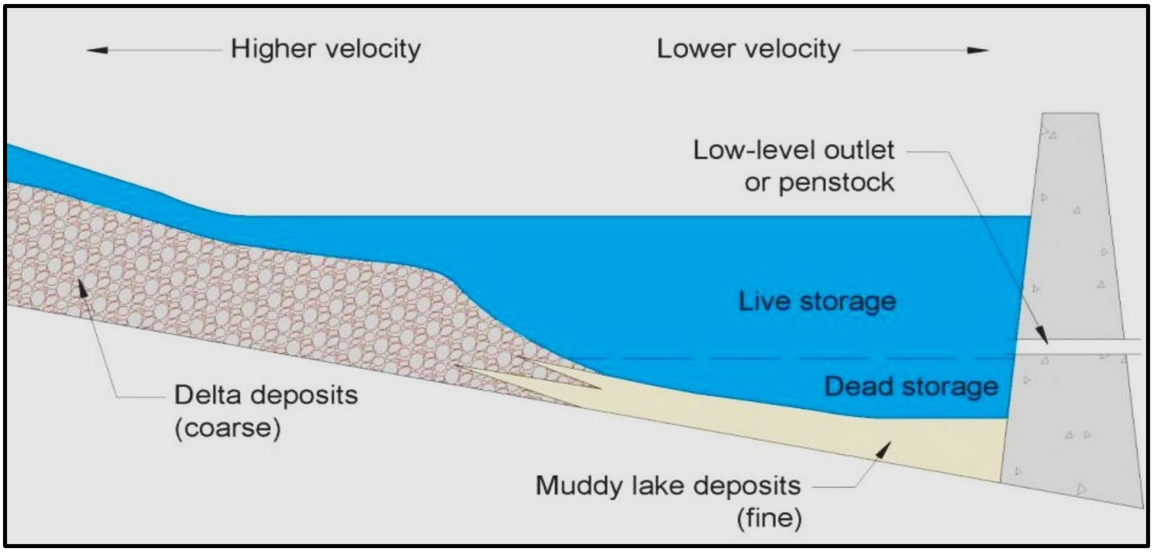

Figure 9: Typical sediment profile in large reservoirs

3. Sediments Interference with Dam Intakes and Outlets

Sediments can often block low-level outlets designed to allow for reservoir drawdown or for normal uses and can clog or otherwise damage gates not designed for sediment passage. During extreme floods, deposition of many meters of material can occur in a few hours. Sediment and debris 17 m deep were deposited in front of Valdesia Dam in the Dominican Republic during the passage of hurricane David in 1979, clogging the power intakes for approximately 6 months.

If sedimentation of reservoirs is allowed to continue, then clogging of spillway tunnels or other conduits may happen. Moreover, reduction of spillway capacity can occur as a result of the loss of approach depth when the sediment front reaches the dam. The reservoir becomes a delta-filled valley that takes a meandering course such that a flood wave does not spread out to allow proper flood routing.

When sediment approaches the dam's outlet works, it can be drawn into pump stations, hydropower turbines, irrigation canals, or other infrastructure. This impact can occur long before the reservoir fills with sediment as these sediments can first be carried into the dam outlet works when the reservoir is partially emptied during the seasonal drawdown for water delivery.

In severe cases, and with absent preventive actions, sedimentation can render equipment such as hydropower turbines or pump stations unusable. In screened intakes, the combination of sediment plus submerged woody debris can clog the outlet or water intake, rendering it inoperable. Because floods transport large volumes of both sediment and woody debris, this clogging can occur quite suddenly after a strong storm, especially if it was preceded by years of steady sedimentation that went unattended and monitoring was absent, perhaps even unnoticed.

The case of Paonia Dam, Colorado gives a good example to this case. The root cause of the problem at the reservoir had been building up since the dam was finished in 1962. Year by year, sediment quietly collected on the reservoir bottom, gradually raising its floor. Once the sediment was level with the dam outlet, where water is released downstream, any debris that washed into the reservoir threatened to clog the opening and make the dam inoperable. In the fall of 2014, personnel worked 10-hour days for two weeks to clear logs, branches and dirt from the outlet by hand and with an excavator. Some of the workers stood directly on waterlogged sand, digging out the grates with pitchforks. When the dam was newly built, they would've needed a crane 70 feet tall to reach the same spot, Figure (10) [16] and [17].

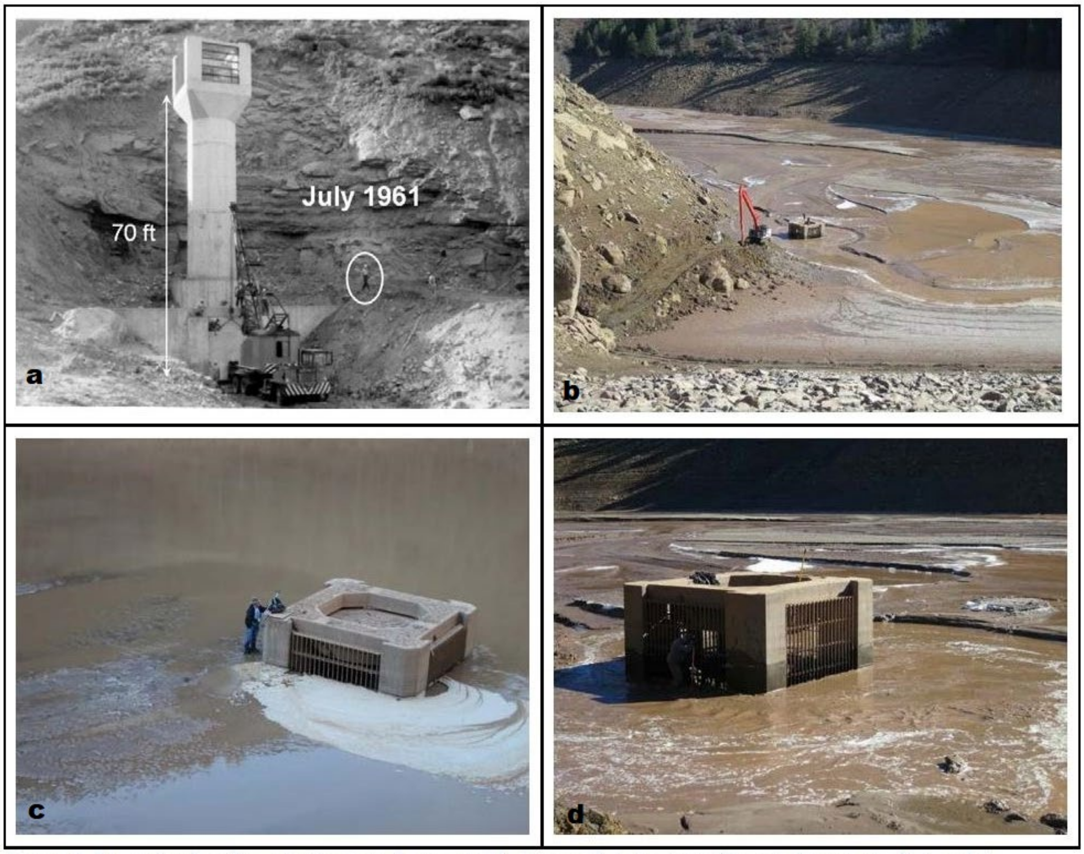

Figure 10: Paonia Dam, a) Intake structure in July 1961, b) During 2014 q long reach excavator was used to clear wood and sediments, c) and d) Sediments level 3 ft higher than inlet bottom sill level.

4. Abrasion Impacts on Dams' Concrete Works

Sediment load and debris can have abrasive impacts on outlet works, mechanical equipment in dams and hydropower stations causing a wear and tear on concrete and metal parts and may cause extensive damages. These generally would not cause total failure but can impair the function of the structure and require intensive costly repair works, while such repair works interrupt its operation causing additional economic losses. Abrasion results from both concrete with low strength and poor aggregates, and from design related problems, so that debris and sediments carried by the strong water current to the downstream sweep back into spillway and outlet works stilling basins resulting in particles abrading the surface in a roller-mill fashion. The hydraulic jump sections of those stilling basins where turbulent flow conditions occur, are particularly vulnerable to abrasion damage. Even the best concrete cannot withstand this wearing action for very long time and such damage results in disintegration of the material exposed to the abrasion mechanism causing additional economic losses. Figure (11) shows the abrasion erosion damage that occurred to the dentate, of the Yellowtail Afterbay Dam sluiceway stilling basin.

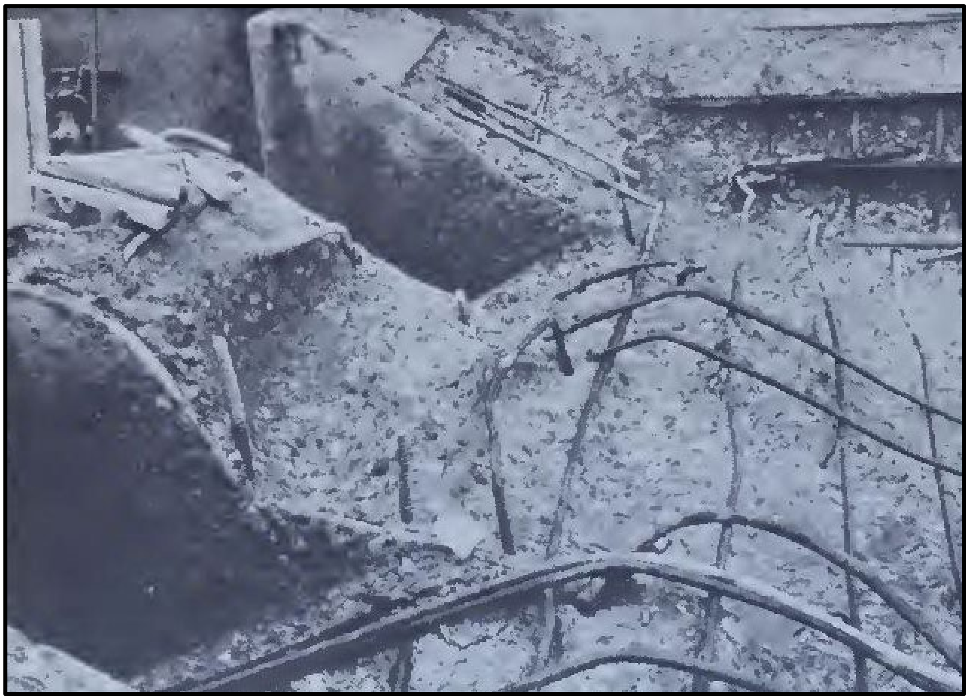

Figure 11: Abrasion erosion on concrete in the dentates, walls, and floors of the Yellotall Afterbay Dam Sluiceway stilling basin. The "ball mill" action of cobbles, gravels, and sand in turbulent water abraded the concrete, thus destroying the integrity of the structure.

Characteristics of this type of erosion are the badly worn reinforcing steel and aggregate as shown int in this figure. In Figure (12), the abrasion effects of coarse sand particles on the spillway at the Milburn Diversion Dam near Sargent, Nebraska are clearly shown, and Figure (13) is a close up photograph of this abrasion [18] and [19].

Although most severe cases of abrasion damage occur in the areas just described, similar damage could be expected in diversion tunnels, power tunnels, canals, and pipelines carrying wastewater.

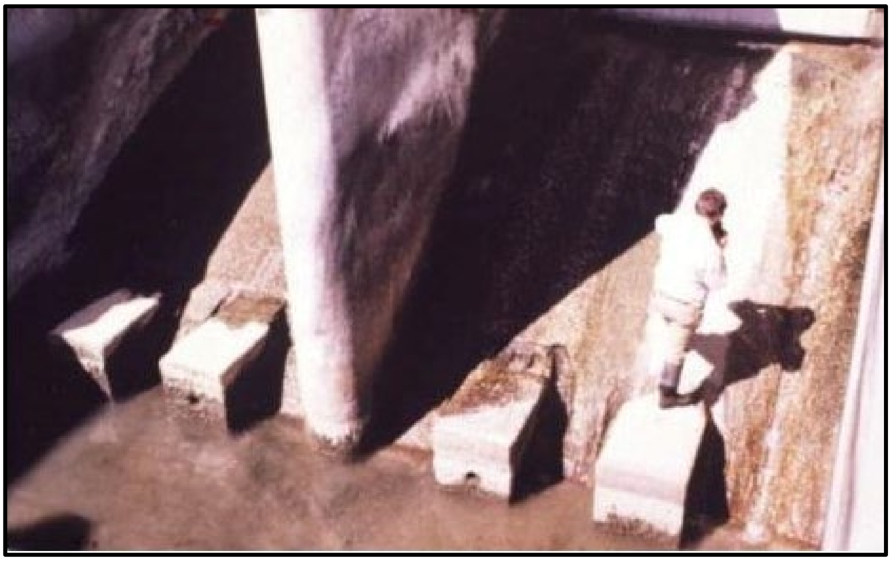

Figure 12: Sand has abraded the spillway at the Milburn Diversion Dam near Sargent, Nebraska [18].

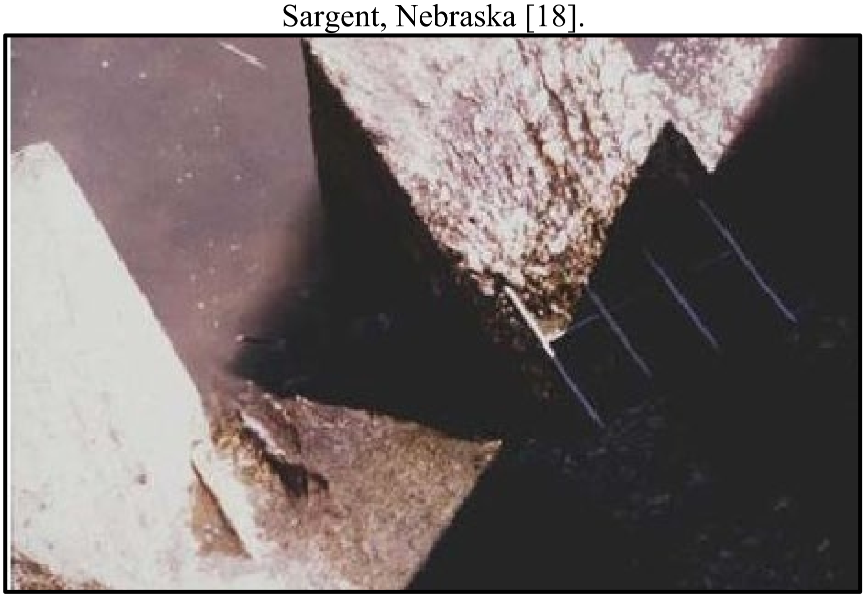

Figure 13: Close up photograph of sand abrasion of the Milburn Diversion Dam Spillway near Sargent, Nebraska [19].

In many cases major repairs or rehabilitation is required. One example of such work was the repair of abrasion erosion damage to a spillway water diversion structure shown in Figure (14). After surveying the extent of damage, it was ascertained that the cause of damage to be abrasion erosion and, after cleaning, it became apparent that the damage was much more widespread and more serious than originally thought. In this particular case, concrete was easily removed to a significant depth over most of the spillway. Thorough investigation was conducted by obtaining and examining concrete cores from several areas, and it was only possible then to determine the best treatment program [20].

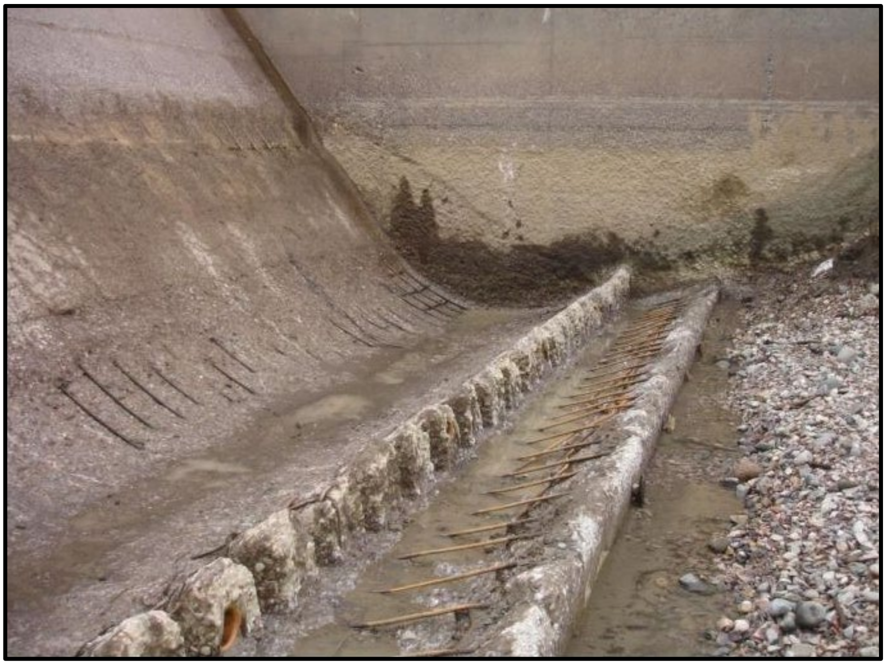

Figure 14: Abrasion erosion damage to the flip bucket and energy dissipaters.

The underlying cause of the damage was not determined prior to starting repairs Many others such abrasive impacts are reported in an article by the authors titled “Dams Safety and problems in Aging Dams” (in the print).

5. Sediment Impacts on Hydropower Equipment’s

In hydropower facilities, sediments coarser than 0.1 mm will greatly accelerate the erosion of turbines runners and Pelton wheel nozzles; even smaller grain sizes may cause damages if containing quartz. This may be the main siltation problem for high head hydropower plants. Also, sediment concentration and total head are important factors. All these will cause reduced power generation efficiency and require removal of generating units from service for repair.

Sediment can damage turbines and other mechanical equipment through erosion of the oxide coating on the blades, leading to surface irregularities and more serious material damage. Sustained erosion can lead to extended shutdown time for maintenance or replacement. Lost revenue and cost of repair can amount to sizable losses.

Factors which determine rates of mechanical abrasion are sediment type and physical characteristics. Angular sediments composed of minerals with a Mohs hardness greater than 5 such as quartz, feldspar and tourmaline are problematic. In addition, hydraulic and facility operation parameters such as flow velocity, hydraulic head, turbulence, turbine rotation speed and turbine material affect abrasion susceptibility.

Plants often are designed to remove most of the coarse sediment particles. However, even silt can cause significant abrasion if the quartz content and pressure head is high enough as mentioned already. The 1,500 MW Nathpa Jhakri hydroelectric plant in India used four desilting chambers that were successful in removing coarser sediments. However, damage from the finer particles was so severe that parts of the turbines had to be replaced within one year.

The other case of Khimti hydropower plant, Nepal (KHP) is a good example of damages caused by sediments. The plant has a gross head of 684 m between the intake at Palati on the Khimti River at an elevation of 1272 m (a.s.l.) and the outlet into the Tama Koshi River at an elevation of 586 m (a.s.l.). Minimum discharge of 500 l/s is released in dry season to maintain downstream water requirements. This plant has 60 MW installed capacity and approximately 350 GWh annual energy production through five Pelton units of 12 MW each with a rated flow of 2.15 m3/s. The power plant has been in commercial operation since July 2000. The effect of sediment has already appeared in runners, and needles and spare runners have already been changed in all units. The damage in the turbine components was inspected in July 2003. After about 6000 hours of operation, a significant amount of erosion had appeared in the turbine buckets and needles. Even though the settling basin with the installed Sediment Sluicing Serpent System performing satisfactorily, large sediment loads with particles smaller than the design size were passing through turbines. Due to the fact that those were high head turbines, the high sand and slit particle impact energy of approximately 8.12μJ (which is relatively high) were severely eroding the turbines components. The needle and the bucket of the Pelton turbine were damaged due to erosion. Erosion had destroyed the jet and reduced the performance of turbines. The bucket thickness was reduced by about 1 mm towards the roof of the bucket, which is critical from the point of view of strength and hence the reliability of the component. Similarly, the splitter of the bucket was eroded to saw tooth form from the original straight edge. The sharp edge of the splitter had blunted, and the width became approximately 4 mm. This width is 1% of bucket width 1% loss of relative efficiency can be expected in these runners. In a 60 MW power plant, this is a significant loss of revenue. As well, the maintenance costs also increases drastically. The owning company tried to minimize the effect of erosion by spraying hard ceramic coatings on the bucket and needle surface at the cost of around US$ 25000 per runner, but this is not that promising.

The Khimti hydropower project in Nepal represents a typical high head power plant in the Himalayan River that is affected by sediments [21]. In Figure (15), a photograph is showing sedimentation abrasion on the turbine runners of the high head power plant; the Jhimruk hydropower plant in Nepal also [22].

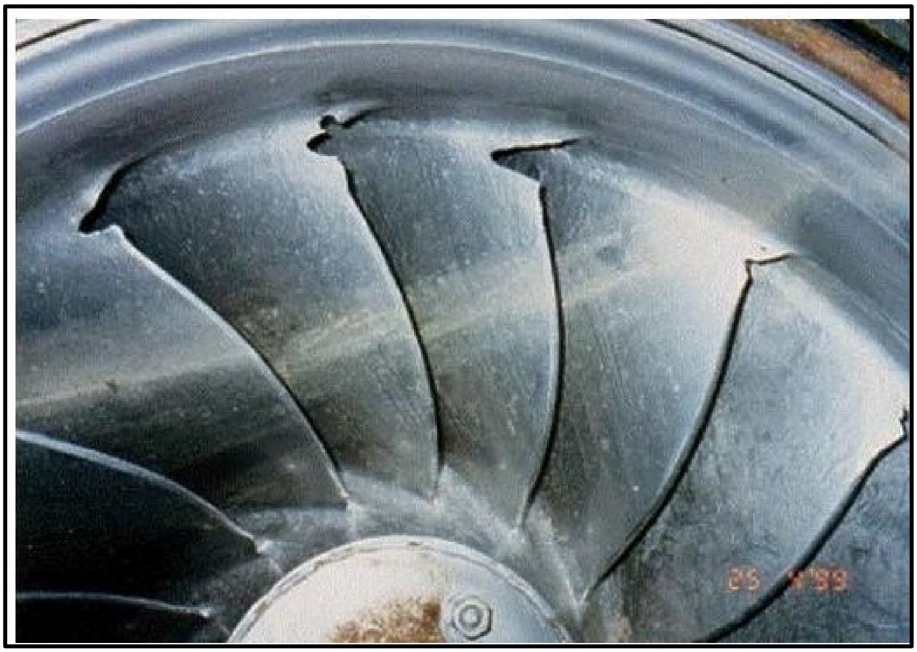

Figure 15: Sedimentation abrasion of the turbine runners at the Jhimruk hydropower pant, Nepal [22].

6. Problems with Gates and Trash Screens due to Sediments and Debris

Mechanical control equipment's such as gates, sluices and associated trash screens can suffer greatly from sediments and debris carried by floods. In the case of Valdesia Dam in the Dominican Republic, mentioned in paragraph (3), is a clear example of the consequences of their impacts.

It was during September 1979 when this dam, located in the Dominican Republic was hit by hurricanes David and Frederick within the space of several days. The radial gates were not opened before the storm knocked out both sources of power supply, and the unopened gates were overtopped for 4 days and were destroyed. Large amounts of sediment and debris, including full-grown trees, were washed into the reservoir during the storm and became lodged against the dam, blocking the bottom sluices. Soundings revealed sediment and debris accumulations up to 17 m deep within 1 km upstream of the dam. Cranes were used to grapple with material away from the bottom sluice, but even though the bottom sluice began to flow it would become clogged again by more woody debris within a matter of hours. After several months of work, the attempt to reopen the bottom sluice was abandoned. Dredging was undertaken to remove the accumulated sediment, using a 700-mm siphon dredger, with a hydraulically operated basket tooth-type cutterhead. This dredger, donated by the Italian Government in the form of international aid, was the largest siphon dredger ever employed in a reservoir. The discharge line was connected to a circular sluice drilled through the base of the dam; the hydrostatic head thus discharged silt through the pipeline without the aid of a pump.

Dredging began in November 1988 and continued on a regular basis for 11 months and on an intermittent basis for the following year. According to the dredger operator, there was a relatively little problem with clogging by woody debris. On two occasions, woody debris clogged the pipeline, which was uncoupled to remove the obstructions. The cutter head itself was clogged on a number of occasions, and in each case, the cutter head was lifted, rotation reversed, and woody material removed by hand. Clogging problems were encountered only in limited areas of the reservoir most affected by the hurricane-driven debris. The dredged slurry was discharged directly to the river downstream of the spillway. The dredged sediments had accumulated in the stream channel below the dam to a thickness of over a meter and were covered with both wetland and upland vegetation. Because the power penstock discharges more than one kilometer downstream of the dam, dredged sediments accumulated in the normally dry riverbed below the dam. These sediments would be scoured and washed downstream only during a flood that produced spillway discharges [23].

Damaged sluice gates by sediments and debris can also compromise the safety of dams when passing high floods. This can be illustrated by the damage that inflicted the sluice gates of Waihi Dam in New Zealand. Waihi Dam located 26 km north-west of Wairoa, New Zealand, was heavily damaged during a severe weather event in September 2015. As a hydropower dam, it was normally supplying 1600 homes with power. The dam’s sluice gates which were supposed to pass the sediments and debris carried by floods were damaged, as a result from silt and debris being released into the Waiau River system resulting from the storm. The storm that hit the Waihi Dam catchment area in late September 2015 was a significant and severe weather event when 196 mm of rain was recorded on 20th and 21st September. The amount of sediments and debris that were carried by the river were unprecedented.

The damage to the dam’s sluice gates was unprecedented and entirely unexpected. Prior to the September 2015 event, and since Waihi Dam commissioning in 1999, no problem was experienced with the operation of the sluice gates. In this event, considerable debris travelled down the Waihi stream, overwhelming the log boom for the dam, and a large amount of sediment and debris was pushed up against the dam. In all previous storm events and times of high inflows, the gates have operated as intended by releasing any sediment and debris during periods of floods, and nothing had built up behind the gates.

From data recorded up to 2010 the sluice gates have operated up to 10 times per year, though some years less infrequently. In this particular event, the nature and extent of the damage to the gates were not immediately apparent in the days and weeks following the storm, but the operation of the three sluice gates following the storm was closely monitored due to the importance of the sluice gates to the safe operation of the dam, and it was anticipated that without properly operating the sluice gates accumulation of such sediments and debris can compromise the structural integrity of the dam structure.

Moreover, the following actions were undertaken in the following days.

(i) with respect to Gate (3), it was opened while the adjacent gate (2) was closed in an attempt to dislodge sediment/debris around Gate (3),

(ii) with respect of Gate (2), it was opened and closed also in an attempt to remove what was believed to be debris caught in this gate. Grappling hooks were used to try to dislodge any debris that was caught upstream of the gate.

A clearer understanding of the possible damage to all gates was obtained after underwater dive inspection on 3 - 4 November. Having established that the safety requirement to have at least two operational gates was compromised, the engineers worked with specialized diving company and agreed what remedial works should be carried out on the gates. A plan was developed to remove out the gates, fully repair them and return them to work. This plan was finalized on 9th of November, and steps were taken to get the necessary experts onsite to commence the work as soon as possible, which was completed successfully.

It cannot be overstated that it was essential for the dam safety to have the sluice gates always in proper operational conditions. Therefore, the owners of the dam and powerplant included in a regular operation and maintenance program the following measures among other things:

(i) Regular inspections of the gates by specialist divers;

(ii) Periodically opening the gates during times of high flow; as permitted by the agreed operation rules of the dam, which allows any sediment or debris that builds up behind the gates to be cleared;

(iii) Conducting regular, five yearly, deformation surveys; and so, the last survey was completed in March 2014.

Moreover, the owners actively managed sediment within the reservoir by implementing a generation and production regime that primarily utilizes the top 1/3 of water in the reservoir. This regime helps ensure that any sediment deposits are undisturbed by the use of water for generation [24].

The other mechanical parts that are subject to possible damage by sediments, and debris are trash screens used to protect intakes from abrasive damage.

The problems of the trash screen in intake structures stem from the fact that they are intended to stop the entrance of debris and large pieces of sediments from going through the structure and impair its use, but in this process, the bigger sized debris may get stuck in the trash screen openings allowing the accumulation of more trash of smaller sediment particles and trash and debris.

If an inspection reveals a flow restriction due to trash, sediment, animal activity, or vegetation, then the obstructive material should be removed immediately. Maintenance procedures should call for more frequent inspection by operational personnel, and more frequent trash and sediment removal, and if the anticipated trash load based on use of the reservoir may need re-evaluation, then the installation of new trash screens of better design or improvement on the existing trash screens may be warranted. For example, if large debris is getting between bars of the trash screen, then a trash screen with smaller openings is probably needed. On the other hand, if relatively harmless debris such as grass is clogging the structure, a trash screen with larger spaces may be more appropriate. Soil conservation measures upstream of the dam may need evaluation if excessive silt or debris is accumulating in front of the dam are causing such problems.

One rehabilitation work to trash screen is exemplified at Jor reservoir low level outlet. During the reservoir draw down in 2007, the reservoir outlet experienced serious blockage due to entry of large quantity of sediments, logs and debris. Further investigation revealed that the trash screen above the reservoir outlet failed to sieve off the large incoming logs from the upstream end. It was suspected that the trash screen bars might have been damaged and dislodged creating a large opening at the trash screen thus permitting larger logs to enter the reservoir outlet. Hence, the rehabilitation works on the trash screen was recommended to be carried out urgently to enable the continuous use of the outlet valves without further blockages.

Field investigation works were carried out prior to the rehabilitation works; and the following was included;

(i) Bathymetric survey and estimation of sediment depth and volume above the reservoir outlet;
(ii) Soil investigation for sediment characteristic profiling;
(iii) Physical dam safety inspection to determine any further signs of damage;
(iv) Reservoir outlet testing to confirm discharge capabilities, and;
(v) Dam instrumentation monitoring and interpretation.The scope of rehabilitation works included;
(i) Carry out Pre, Interim and Post Hydrographic Surveys to continuously monitor the sediment build up;
(ii) Design and construct a complete set of dredger pontoon equipped with the working platform, lifting equipment, mechanical pump, agitators/cutters;
(iii) Carry out de-sludging/dredging works to clear the sediments, trash, debris, tree branches, etc. deposited on top of the existing trash screen;
(iv) Carry out underwater physical inspection;
(v) Design, fabricate and install a new high tensile trash screen above the reservoir outlet area, and;
(vi) Design and fabricate a complete set of turbidity/silt curtains as a separator and as a sediment control system.

The design of the trash screen included provision for overburden loads and pressure from the deposition of sediments and trash, logs, debris, etc. The loads were calculated based on the characteristics of the sediments depths and volume deposited above the reservoir outlet. The new trash screen was designed to be fabricated at factory and installed underwater directly on top of the existing trash screen. Suitable factors of safety were adopted in order to determine the most practical depth of sediment deposition expected for a period of 50 years. The metal works installation included:

(i) High tensile, hot dipped steel trash screen including all necessary fittings to be installed above the existing trash screen at the reservoir outlet;
(ii) The new trash screen was to be fully assembled at factory and be installed directly on top of the existing trash screen by drilling;
(iii) The new trash screen mobilization and installation works was designed to be carried out entirely underwater.

The most important criterion in this reservoir, as in all other reservoirs in similar situations, is to retain the reservoir outlet structure in accordance to the dams safety standards [25].

7. Impacts of Sediments on Dams Stability

In most design standards of dams, silt load is taken as a horizontal force acting at hs/3 distance from the bottom; as in Figure (16) and its magnitude is calculated from:

$$P_{silt} = 0.5\gamma_s h^2 k_a$$

Where:

k_a: coefficient of active earth pressure of silt which equal to $\frac{1 - \sin\phi}{1 + \sin\phi}$

ϕ: angle of internal friction of soil, cohesion neglected.

γ_s: submerged unit weight of silt material.

h: height of silt deposited.

Idealized representation used for design of Dams assumes a uniform deposition of sediments as indicated in Figure (16).

Actual conditions in reservoirs are not exactly the same as shown in this figure. No silt is deposited when the dam is new, but after some months or years, silt is deposited at the bottom and accumulates at the dam face which may take a configuration far from being similar to this idealized picture.

Sediment loads are commonly idealized as a static at-rest soil pressure. For example, the U.S. Bureau of Reclamation’s design manual for small dams suggests that sediments be considered equivalent to a fluid with an implied pressure coefficient (ka) of about 0.39 and an internal friction coefficient (Ø)of about 37º [26].

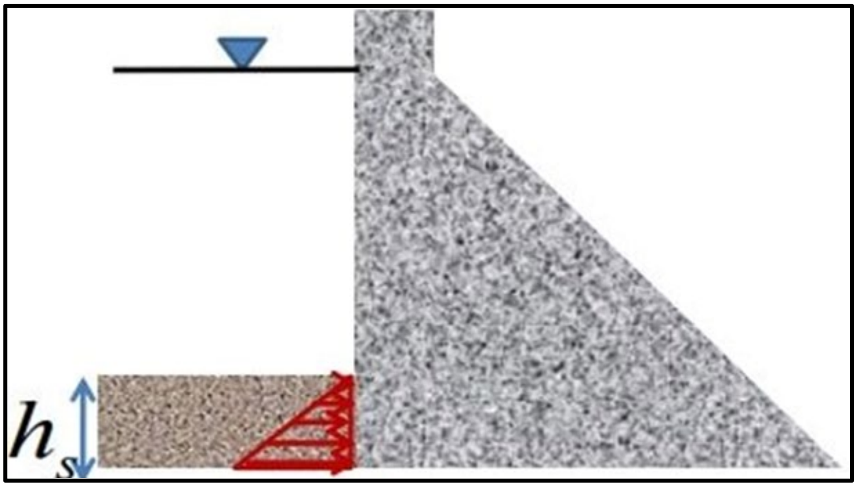

Figure 16: Idealized visualization of silt accumulation and Pressure distribution.

In the application of the equation given, the assumption of constant value of (h) is not accurate as the sediment profile in the reservoir is constantly changing, and so is the accumulation of sediments behind the dam.

Moreover, actual reservoir sediment properties can vary considerably along the reservoir bottom at the dam face. Unconsolidated fine-grained sediments are likely to have lower shear resistance and a higher at-rest pressure coefficient, while a reservoir filled with coarser sediments may have a higher shear strength. A typical reservoir sediment profile as shown in Figure (17) is given by [27] which suggests that the progressive finer materials being deposited as the flows approach the dam.

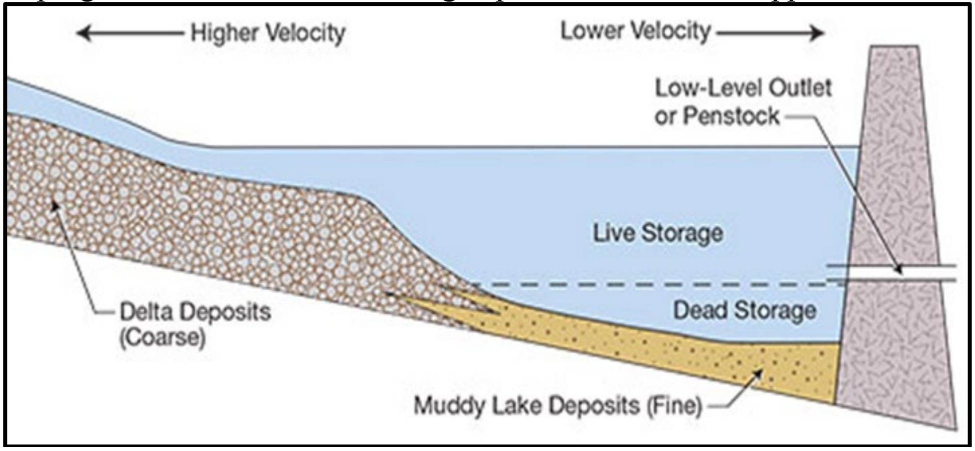

Figure 17: Typical Sedimentation in the reservoir behind the dame takes the form of progressive finer materials being deposited as the flows approach the dam [27].

Published criteria with respect to potential changes in uplift pressures due to sedimentation often neglect the fact that fine grained sediments may reduce uplift in the same manner as does an engineered upstream blanket. Conversely, in the case where there is a large turbid inflow, higher uplift pressures would be expected until enough particles had settled to form a blanket.

Sediments in the reservoir are in constant change and never in a stable form which results from the continuous deposition and delta reworking resulting from hydrological events. One case which explains this point is cited by Morris and Fan (1998) and illustrated in Figure (18) [27].

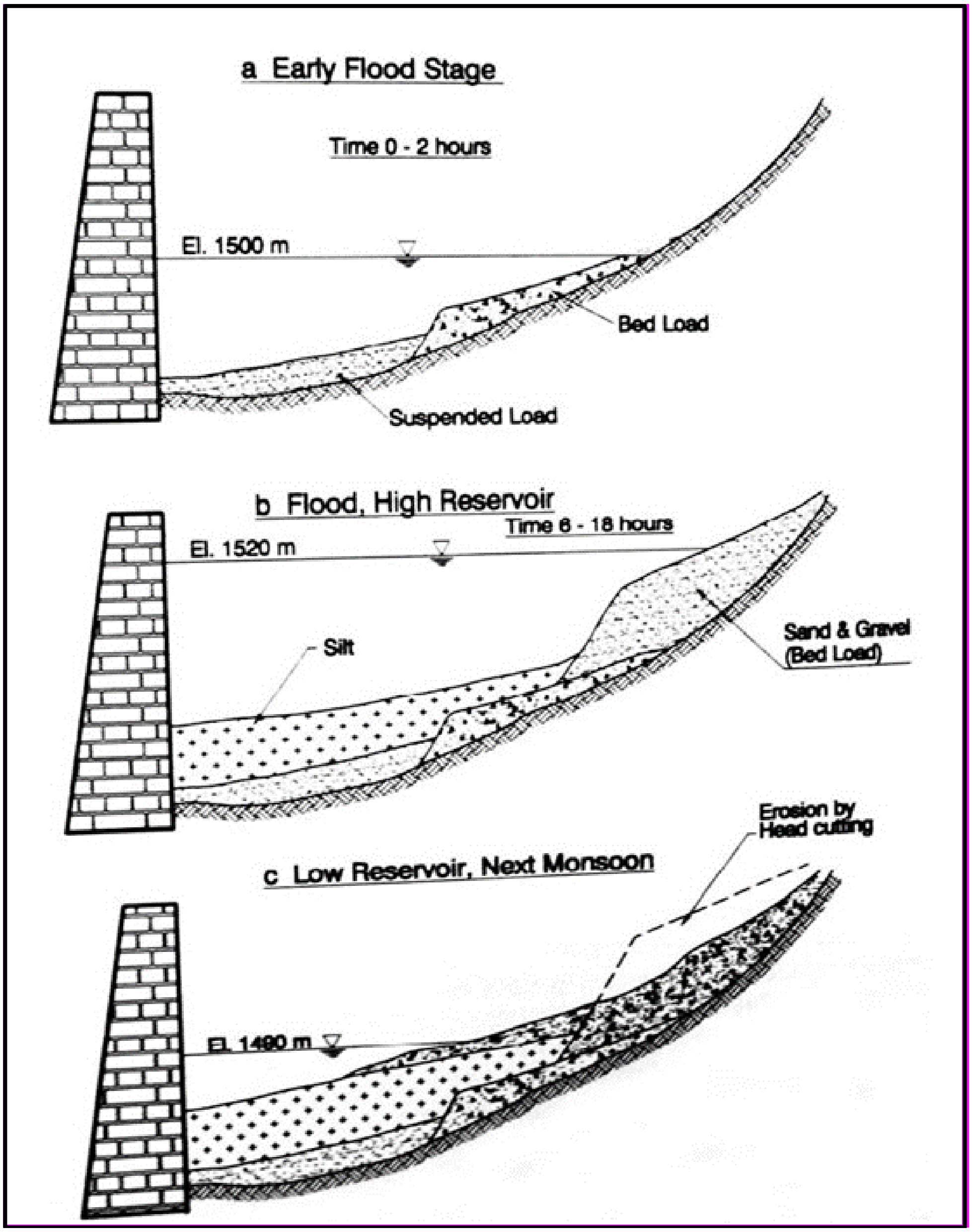

Figure 18: Process of sediment deposition and reworking of delta deposits resulting from a massive sediment event at Kulekhani Reservoir, Nepal after Garry and Okaji (1995) [27].

Commonly used design procedures can omit some plausible load cases. For example, an underwater sediment land slide and slope failure could cause surface waves creating the potential of tsunami wave. Designers need also to consider the potential that failure of the steeply sloped deltaic front could increase loading and produce compression waves that may fluidize finer sediments and creates turbidity currents near the toe of the landslide and ignoring the fact that relates to turbidity currents in reservoirs. Such turbid fluid with a sediment load of 100 mg/l could be about 6% heavier than clear water. As the deposition advances toward the dam, the potential for such issues progressively increases.

Another issue related to dam safety during a seismic event, is the likely liquefaction of sediments, which would quickly return them close to their original state by losing all strength and exert a dense fluid hydrostatic load on the dam. However, this degree of fluidization is not likely possible in a reservoir filled with coarse materials. In dealing with such fluidized dense fluid, it is found that designers often assume that such fluid contributes to hydro-dynamic pressure loading based on Westergaard's formula, ignoring the physical basis for its derivation.

Designs also need to consider the degree of saturation of the sediments. There is minimal system damping under dynamic loading when reservoir sediments are fully saturated. However, significant reductions in acceleration occur when sediments are partially saturated. For rigid foundations, hydrodynamic pressures decrease slightly at the dam base when sediments are fully saturated but increase when partially saturated. Partial saturation will increase the system's response to horizontal ground movement.

Sediment thickness is an important consideration, especially when the sediments are partially saturated. Thin layers result in minimal absorption of horizontal motions, largely due to a relatively high modulus of elasticity and low attenuation coefficient. Other important factors are sediment density, compressibility and pore water pressure.

This dependence of dam stability issues on sediment properties makes a strong case for their measurement and inclusion as part of the design. However, designs are performed before sedimentation are done and the same sediments that are stable under normal conditions and absorb energy at the bottom of the reservoir could liquefy. For this reason, the use of a reservoir bottom reflection coefficient must be logically linked to assessment of the reservoir sediment behavior and ongoing monitoring [28].

From the foregoing, it goes without saying that sediments issues have their impacts on dams' stability and should be considered thoroughly during planning and design stages.

8. Conclusions and Recommendations

1. Large volumes of sediments are trapped by dams causing reduced quantities reaching deltas and sea. At the same time, their accumulation into reservoirs reduces reservoirs' capacities and their benefits by reducing in great many cases the active storage for flood routing, power generation and irrigation and other water uses. This is a disadvantage which has to be understood well from the beginning of t planning stage, and its impacts should be calculated and accounted for in the economic analysis of the project.
2. Siltation may not be an immediate problem for many dams with large reservoirs, but it can still reduce their storage capacities within decades. It can, however, become a key problem within few years for great many dams with small reservoirs. In all cases, preventive measures to reduce erosion in the river catchment may need to be taken. Mitigation actions can include dredging reservoirs, flushing sediments, clearing intakes from debris, and preventing entry of large size sediments and debris by using properly designed trash screens. All these measures add to the costs of maintenance but may be considered mandatory in many cases.
3. Sediments and debris can also cause extensive damage to intakes, outlet structure, gates, turbines and trash screens and power turbines by their abrasive action. Routine inspections and regular maintenance can play positively to reduce the damage and enhance safety of the dam and its appurtenant structures. Operation and maintenance (O&M) manuals should define clear instruction and define required measures, including routine and after floods inspections, regular maintenance and periodic bathymetric surveys of reservoirs to anticipate problems before they can create safety risks.
4. Safety risks created by sediments on dams' stability should be scrutinized during design stage. Changing of silt load on dams after many years of operation due to an increased depth, changing density and changing reservoir morphology in addition to liquefaction potential of sediment accumulations during earthquakes are to be expected during life time of dams. Good and deep study of sedimentation in the future, reservoir shall be given its proper weight of importance, and a good study of these factors has to be included in the planning and design reports of new dams.
5. Good understanding of the silting up process mechanism of any reservoir and changes of its bed morphology with time can give dam owners an insight of how best ways to operate the dam and what remediation actions are required. Performance of bathymetric surveys on a regular basis helps greatly in enhancing this understanding, and such surveys could be prescribed in the (O&M) manuals and taken as a desirable and recommended regular routine.

6. Present rising public awareness of preserving natural environment has led government in recent years to demand the performance of Environmental Impact Assessment (EIA) studies as a prerequisite for licensing any new dam. Any sound EIA study must devote a good part of it to the impacts anticipated by changing the sediment transport regime of the river imposed by the new dam. Such impacts may disturb the fragile and delicate balance of the ecosystems and biodiversity at the downstream reach of the river leading to loss of fish species or other marine types, which apart from blocking their migration route depends on nutrients carried by silts for their food. Scouring river channel bed in the downstream of new dams can threat the stability of existing structures such as jetties and bridges or impair their functions such as for intakes of water supply and irrigation projects. Similarly, in the upstream reach deposition of silts in shallow reaches may create health hazards due to forming wetlands and health problems such as Schistosomiasis or Malaria.

Morris and Fan (1998) (Reference 23) have given a comprehensive summary in table (18.1) of the environmental issues related to sediment and its management in the aftermath of constructing new dams.

The ETA study shall prescribe all necessary measures to combat such issues and go further to recommend measure to reduce the sediment loads carried into the reservoir in the form of better management practices of the catchment area or constructing sediment check dams.

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Dam Safety: Hazards Created by Human Failings and Actions

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Abstract

Dam Safety and dam incidents are treated here looked at from the “Human Factors” perspective. An attempt is made to explore these factors as an important drive in impairing dams’ safety and increases their risks. Distinction is drawn between the “Normal Human Caused Incidents” and the “Extraordinary Human Caused Incidents” together with the description of their root origins and subsequent consequences. The first type includes unintentional mistakes, errors and flaws committed by the operators of dams inadvertently, in addition to negligence, lack of experience or overconfidence. Such failings can happen in manual operation of dams, or through the use of their Supervision, Control and Data Acquisition (SCADA) systems as in industrial control system (ICS). They can occur also due to flaws in software or even in the application of information and communication technology (ICT) in remote control operations. As for the second group; the Extraordinary human factors, they are defined here as those committed by man with the full understanding of their possible damage. They are done purposely for destabilizing dams after thoughtful and carefully meditated decision making process and they are manifested in acts of war, sabotage and terrorists actions. In this modern age, these acts are characteristics of hackers’ attacks on dam(s) operating systems. This is done through the use of cyberspace by the widespread interconnected digital technology with the accompanying advances in the communication technologies. As such, these technologies have made remote control of such systems possible. Not limited to this, dams remain now, as they were always in the past, the obvious targets in wars and conflicts to inflict losses on the

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enemy and to use them as weapons, and for terrorism actions for challenging governments. Examples of the aforementioned threats are described with examples given from real cases to elucidate the dangers involved. Lessons to be learned from these incidents are derived and recommendations are presented to be followed to avoid risky situations.

Keywords: Normal Human Caused Incidents, Extraordinary Human Caused Incidents, SCADA Systems, ICS System, Software, ICT Technology, Cyberspace, Digital technology, Remote control, Hackers, Terrorism.

1. Introduction

While safety hazards of dams can be created by improper planning, faulty design, or result from improper selection of site and dam materials and construction procedures, such hazards may also be intensified by human actions or inactions, and they may be intentional or unintentional throughout the decision making processes and during the operation of the dam. Description of such hazards supported by case histories can help illustrating the nature of the problems involved and help to draw lessons from them. They may also guide in taking precautionary or protection measures. But it is not enough; however, at just discovering the single prime cause of such events if the full lesson is not learned; any study cannot serve its purpose without tracing the chain of actions and interactions leading to the failure or accident. This implies that dams should be treated as systems containing many interacting components and subsystems and that the root of the problem may be hidden in a single action within the chain of events leading to the undesirable end. Or, more often it can mean that one component of the dam system is more critical to dams safety than others. One distinction must be made right at the start between two major types of accidents. The first category concerns what is called “Normal" accidents, or system accidents, which result from human actions; the second type are those which are imposed on dams from outside but also through human actions and shall be called here “Extraordinary” incidents. Normal accidents are inevitable in extremely complex systems, and given the characteristic of the system involved, multiple failure drives interacting with each other can occur, despite efforts to avoid them. One example is operators’ errors, which are quite common problems, while many other failure drives relate to organization; rather than technology or individuals’ actions. It is also common that big accidents almost always have small beginnings. Such events appear trivial to begin with before unpredictably cascading through the system to create a large event with severe consequences. So “Normal” accidents are spontaneous and are related to the nature of the dam and its operation as a system [1].

The “Extraordinary” incidents, on the other hand, are those caused by man for purposely destabilizing dams after thoughtful and carefully meditated decision making process which is manifested in sabotage and acts of war. Brief reviews of both types are given here with some illustrations.

2. Normal Accidents and the Human Factors

No system has ever built itself, and since few systems operate by themselves, and since no systems maintain themselves, the search for a human in the path of failure is bound to succeed. If not found directly at the sharp end, as a "human error" or unsafe act, it can usually be found a few steps back [2].

This was the conclusion reached by Professor Richard J. Holden from the University of Wisconsin- Madison covering professional safety management, which applies to man-made systems; and dams are no exception.

Dams as systems typically include both human and physical elements, and are sometimes referred to as "sociotechnical" systems. To prevent future dam accidents, it is essential that dam safety professionals understand both the physical factors and human factors, and how they contribute to failures or safety hazards.

Physical factors stem from forces and situations imposed on dams by natural events of floods and earthquakes. Foundations and dams' materials create the spectrum of other potential unsafe conditions. But, while these factors can be calculated and quantified to a high degree of refinement, mistakes or bad judgments can still creep into this defeating the purpose of safety considerations. Human factors contributing to the potential for failure can also result from pressures imposed on designers by the requirements of more water and power generation, or by constructors and owners trying to meet time schedules or even by maintenance constraints due to budgeting problems.

In the operation phase, human misjudgment associated with faulty memory, ambiguity of instructions and incompleteness of information play negative roles, while experimenting with untried shortcuts may bring undesirable outcomes. In critical situations, while operating personnel are facing dangerous occurrences psychological conditions; fatigue and emotions can lead to grave mistakes and undesirable ends. Lack of knowledge, lack of expertise and even negligence is more of the other factor that have contributed to some dams' incidents and failures.

Human errors and the underlying primary drivers of failure noted above often lead to inadequate risk management. Inadequacies in risk management may be classified into three types:

- *Ignorance* which involves being insufficiently aware of risks. This may be due to aspects of human fallibility and limitations such as lack of information, inaccurate information, lack of knowledge and expertise, and unreliable intuition. Complexity can also contribute to ignorance.

- *Complacency* which involves being sufficiently aware of risks but being overly risk tolerant. This may be due to aspects of human fallibility and limitations such as fatigue, emotions, indifference, and optimism bias that "it won't happen to me". Pressure from non-safety goals can also contribute to complacency.
- *Overconfidence* involves being sufficiently aware of risks, but overestimating the ability to deal with them. This may be due to aspects of human fallibility and limitations such as inherent overconfidence bias, which results in overestimating knowledge, capabilities, and performance [3].

In modern procedures applied to various systems, including dams, pitfalls in control software may appear in untried situations; similarly, an unexpected failure of one element of the software can cause dangerous conditions. In such cases, alarm signals might be wrongly interpreted, or operation commands are not correctly received. The more complex the system is, the more are the possibilities of wrong interactions. Such interactions can result in large effects from small causes, including "tipping points" when thresholds are reached, and they make complex systems difficult to model, predict, and control. Complexity, generally exacerbates the effects of human fallibility and limitations. An example of this from the aviation industry is the flaws were embedded in the control software leading to the two Boeing 737 max 8 catastrophes in 2018 and 2019. Software known MCAS which was supposed to ensure the plane flew smoothly was expanded in 2017, but the new version installed in the plane was risky as it relied on single sensor that could push down the nose of the plane by a much larger amount. Regulators had never independently assessed the risks of this, which led eventually to these catastrophes of 346 peoples being killed in the two plane crashes.

As far as human factors are concerned, the following six aspects are key observations regarding past failures of dams and other systems:

- Failures are typically preceded by interactions of physical and human factors, which begin years or decades prior to the failure.
- The interactions among physical and human factors are often not simple and not linear. Instead, they may be complex and involve nonlinear relationships, feedback loops, causes having multiple effects, effects having multiple causes, and a lack of distinct "root causes" or dominant contributing factors.
- Interactions among physical and human factors usually generate "warning signs" which are not recognized, or not sufficiently acted upon, prior to the failure.

- Physical processes deterministically follow physical laws, with no possibility of physical “mistakes.” Therefore, failures, in the sense of human intentions not being fulfilled, are fundamentally due to human factors, as a result of human efforts individually and collectively “falling short” in various ways. A story of *why* a failure happened; therefore, cannot be complete without reference to contributing human factors.
- A natural tendency is for systems to move towards disorder and failure, in line with the concept of increasing “entropy” in physics. Therefore, systems such as dams are typically not inherently “safe,” and continuous human effort is needed to maintain order and prevent failure.
- Systems such as dams, including the people involved in designing, building, operating, and managing them, tend to conservatively have numerous “barriers” which must be overcome for failures to occur. This generally makes failures unlikely and results in low overall failure rates. However, when dealing with a large number of systems, such as the approximately 90,000 dams in the United States, it can be expected that “unlikely” failures will sometimes occur, due to physical and human factors “lining up” in an adverse way that overcomes all barriers [4].

3. Dams incidents Caused by Human Factors During Operation

3.1 General

Dams have been operated traditionally by trained personnel some of whom are residing on site, manning all operations from a control center located within the site. Some of the operating team are always on duty, and this will depend on the size and importance of the dam. In case of a sudden event, such as an unexpected strong storm or an earthquake, then immediate safety actions can be taken such as opening spillway or outlet gates to relieve the pressure on the dam or to call for help from outside to support the effort and to declare emergency situation if needed and even sound the alarm for starting an evacuation effort for the threatened communities downstream. Moreover, any malfunctioned control equipment during this event would be put right in time, and bad consequences can be averted. All this sound as good and well thought of practice, but many dam failures have resulted either from bad management during emergencies, or through negligence and carelessness or from overconfidence and short sight caused by lack of knowledge and false confidence that nothing adverse will happen.

The same thing occurs from having the notion that whatever happens it can be mastered and solved in time. With the progress being made in technology, and in no doubt under economic pressure, new technologies have been introduced in the control and operation of dams. It is assumed that the new technologies can bring the same degree of safety, if not better, as manual operation. Advances made in modelling and simulation techniques and software advances, in addition to high tech communication systems, have given a higher confidence in this field. Ironically, these same technologies have also contributed to some incident and failures in the same way as what had happened in the Boeing 737 Max 8 cited before. In operating a network of dams, whether in flood control or power generation, the control is left to a single or very few persons tending to this by remote control operation systems; while sitting in remote control centers away from the dam site(s). Decisions are made depending on a Supervision, Control and Data Acquisition (SCADA) systems without directly seeing the structure (s). Any fault creeping into these systems, or any wrong interpretation of the data streaming or recorded by the monitors, can lead to an incident or failure which may go undetected for an additional time leaving no chance to take any quick remedy. Similarly, spillway gates are now seldom operated by a dam tender going to the dam crest and pushing a switch that directly allows operation of the gate motor. Now, a remote operator may click a virtual button on a computer screen. In the first case, the dam tender gets immediate visual feedback that the proper gate is indeed moving or not. In the second case, the remote operator gets a signal that the gate is moving from some form of position sensor. If the sensor is giving erroneous data, the operator has no real knowledge if the gate is moving or how far it is moving. The loss of positive confirmation of gate position would result in a gate being raised far in excess of the operator's intentions and larger than intended release of water causing damage in the downstream reach. In the following, some examples of recorded cases are given.

3.2 The Euclides da Cunha Dam case: An Example of Over Confidence

Euclides da Cunha Dam in Brazil was built in 1958 and failed on January 19, 1977.This dam was 40 m high and 300 m long, and stored 25 Km^3 of water. For a catchment area of 4300 km^2, the gated spillway capacity was 3000 m^3/sec. The inflow was calculated to be 2000 m^3 /sec during the event of a storm, but the two radial gates were kept closed due to human error when the operating crew stayed for lunch and when returned, as a cause of heavy rainfall, the access road had become impassable. The embankment withstood seven hours of overtopping up to one meter until it breached, and the breach width was limited to 100 m. The maximum discharge of flood from failure, which was in the range of 10 000 m^3 /sec, caused the failure of the dam. The flood destroyed also the 41m high Annan does de Salles Oliveira earthfill dam at the downstream. However, there were no fatalities. The dam was rebuilt with an additional (free-flow) spillway. This is a case of overconfident operators of nothing out of the ordinary can happen during their absence from the site, and if anything happen, then they can control it, both assumptions proved to be wrong. While the overconfidence of the designer, that nothing of this sort was possible, resulted in brushing aside the need of the extra safety of a free flow emergency spillway [5] and [6].

3.3 Failure of Dibis Dam; An example of Negligence and Bad Management

This dam is located on the Lesser Zab River in Iraq approximately130 km upstream from its confluence with the Tigris River. The purpose of the dam is to divert water from the Lesser Zab River into the Kirkuk Irrigation Project main canal. It was constructed between 1960 and 1965 as part of the larger Kirkuk Irrigation Project for the irrigation of 300,000 hectare of very fertile land. Dibis Dam as designed is a 23.75m high earthfill dam made of gravel-alluvial fill material and concrete diaphragm central core, combined with concrete section forming the gated spillway structure. Figure (1) shows the spillway of this dam.

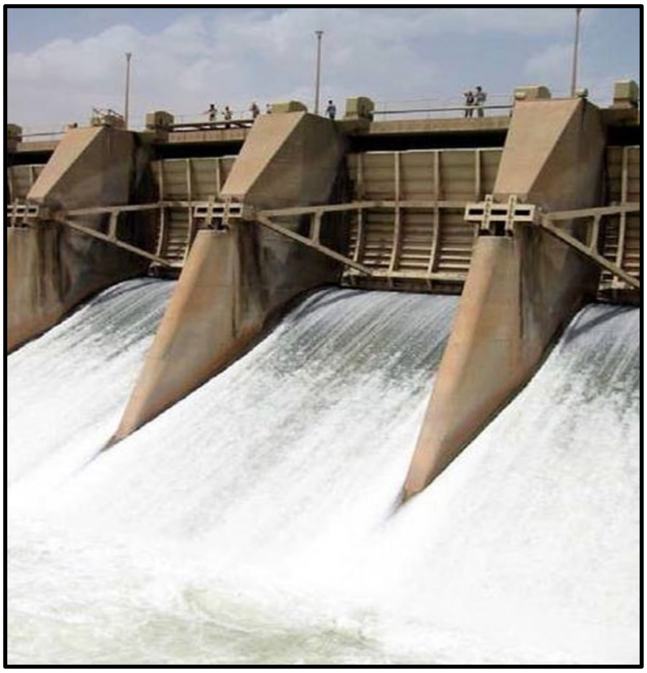

Figure 1: Dibis Dam in Iraq [7].

The spillway has a capacity of 4,000 m^3 /sec through the gated structure while additional discharge of 278 m^3 /sec can be passed through the head regulator of the Kirkuk Irrigation Project which is located at the upstream right side of the dam. Inflow to the dam is from the upstream Dokan Dam releases and from any flow discharge that originates from the intermediate catchment between the two dams which contribute considerable runoff during heavy rains.

On the night of 1st of March 1984, the dam failed during an intensive rain storm which coincided with large inflow from Dokan Dam. The fuse-plug that should have worked in such rare occurrences did not erode; the local authorities d constructed a concrete slab for road and for passing large diameter water supply pipe. The operator had left the site to spend the night in his home in Kirkuk with the gates partially open without leaving replacement to operate the spillway radial gates in case of emergency as the one when the flood occurred. Combination of a high flow release from Dokan Dam with remarkably high runoff from the intermediate catchment caused the overtopping and erosion of the earthfill embankment completely leaving the spillway intact. The fuse plug intended to erode to prevent failure in such a case did not work.

Unofficial sources put the number of fatalities at nine. The dam operator was charged with manslaughter and sentenced to life imprisonment, while the engineer in charge of dam administration stationed in Kirkuk was sentenced to nine years imprisonment for lack of attention and faulty management, in addition to the Director General of the State Organization for Dams and Reservoir (SOD) in Baghdad being dismissed. The dam was rebuilt again between October 1985 and March 1987 [7].This failure is a clear case of negligence, which aggravated a situation already compromised by other failings and bad management. Leaving the site unattended by the operator without permission was only the final mistake in a series of mistakes. The dam engineer, living in Kirkuk and not on the dam site, could not have paid enough attention to dam safety issues in such a situation or in any other risky situation and even lacking the ability to provide operators replacement if needed. Failure of (SOD) in Baghdad, to provide enough residence facilities at the site could have contributed to this. Moreover, allowing the construction of concrete paved road and laying of large diameter water supply pipe on top of the earth embankment, with or without knowledge of the Engineers office or this organization, was the most hideous and outrageous act of ignorance, which nullified the fuse plug function that could have stopped crest erosion FAILURE as intended by the designers. The Consultant of the project should have, in any case, put special emphasis on this matter in the project (O & M) repot, but he did not do. The absence of a dependable communication system between the two dams, the dame engineer office and with the main office in Baghdad is one more management gap that otherwise could have stopped this event by ordering the Dokan Dam office to reduce or even stop altogether the release from that dam.

3.4 Nimbus Dam Incident: A case of Technological Failure and Human Interaction

The Nimbus Dam was completed in 1955 and measures approximately 75 feet high and 1,090 feet in length. The dam serves as an afterbay structure for Folsom Dam to reregulate flows of the American River for flood control, and as a diversion dam to direct water into the Folsom South Canal, while at the same time serving as a forebay for the hydroelectric generation station. Nimbus Dam includes two generators capable of producing more than 15,520 kilowatts of power. As a regulating reservoir, variations in water levels on Lake Natoma occur daily, but are generally only between two and four feet. Flow control is accomplished through 18 radial gates with individual gate bays as shown in Figure (2) [8].

A case of hazardous situation generated by technological failure had developed in this dam during operation routine, but it was relieved by human intervention, and therefore, is worth mentioning.

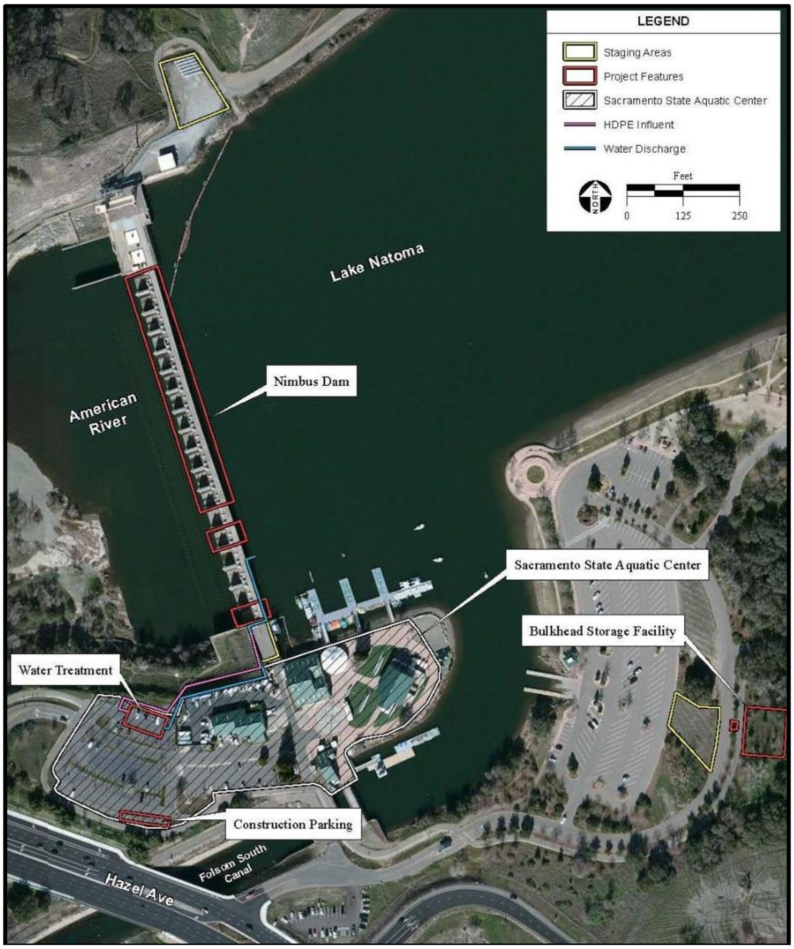

Figure 2: An aerial photograph of Nimbus Dam, [8].

The exact scenario of the incident was played out at the Nimbus Dam on one Saturday of February 2006 when a loose electrical connection on one gate broke as a result of gate vibration caused by the water gush. As a consequence, this resulted in the failure and malfunctioning of the respective control sensor and caused the gate to open for 30 minutes raising the water level in the river by 5 ½ feet. Fortunately, this happened with only one gate, and for thirty minutes the gate stayed wide-open. It was the first serious malfunction in the gate control system, which was installed about two years before.

The problem occurred at 1 p.m. Saturday as four of the dam's 18 gates were being opened slightly to increase the flow of water downstream of the dam from

5,500 to 7,000 cubic feet per second. The malfunction caused one of the gates to fully open, increasing the flow to about 20,000 cubic feet per second.

One of the observers comments on the incident was reported as saying: “When you raise the gates, there's a sensor that is supposed to kick in and stop the gates from opening beyond a set point, when the sensor failed, one of the gates continued to open”.

According to a U.S. Geological Survey chart, a flow increase from 5,500 cfs to 20,000 cfs raises the river's average depth from 7.2 feet to 12.7 feet.

The gate remained open for about 30 minutes before an operator closed it using a manual override switch on top of the dam. The loss of positive confirmation of gate position resulted in a gate being raised far in excess of the operator’s intentions and in larger than intended release of water causing the stranding of people in the downstream reach. But human interaction at the right time had stopped a technological failure from creating a very hazardous situation [9]. This case illustrates a positive human interaction which has stopped flooding of the downstream and highlights the vulnerability of technology in systems such as dams.

3.5 The Taum Sauk pumped storage plant Failure: An Example of ComplexInteractions of Human Factors, Technological Errors and Society Demands

The Taum Sauk pumped storage plant is a power station in the St. Francois mountain region of Missouri, United States about 140 kilometers south of St. Louis near Lesterville, Missouri [10].The plant was constructed from 1960 –1962 and was designed to help meet daytime peak electric power demand. It began operation in 1963. The plant consists of a lower reservoir, which is sited along the East Fork of the Black River, and an upper reservoir, which is formed by a kidney-shaped rock-fill dike approximately 15.2 to 26.5 meters high, capped by 3.05 meters concrete parapet wall set on a crest that is 3.66 meters wide.

The upper reservoir held 5.67 million cubic meters when filled. A variety of design/construction flaws, an instrumentation programming error together with other human errors contributed to the failure of the upper reservoir on December 14, 2005.

Malfunctioning and improperly programmed, and placed sensors failed to indicate that the reservoir was full and did not shut down the facility's remaining pump unit water had been overflowing for 6 to 7 minutes. This overflow undermined the parapet wall and scoured the underlying embankment, leading to a complete failure within that time frame. The peak discharge from this outbreak flood was estimated to be 8,184 m^3/s, obliterating most of Johnson Shut-ins State Park, where, miraculously, only five people were injured. The flood pulse was significantly mitigated by capture within Lower Taum Sauk Reservoir, and the maximum discharge over the Lower Taum Sauk Dam was limited to just 45.3 m^3/s, precluding any significant downstream damage [11] and [12]. The photograph in

Figure (3) taken below panel 71-72 of the parapet wall shows the deep plunge pool that developed and the subsequent undercutting of the wall, while Figure (4) indicates arrows representing the outflow nape once it extended beyond the wall footing and directly onto the underlying rockfill. The rate of scour and erosion increased dramatically once this occurred.

The failure of Taum Sauk was not due to a single easily identified cause that was initiated on the fateful day. The failure began the day the project was conceived, and the failure was the result of many seemingly unrelated decisions and systems that interacted in complex ways that were not anticipated [13].

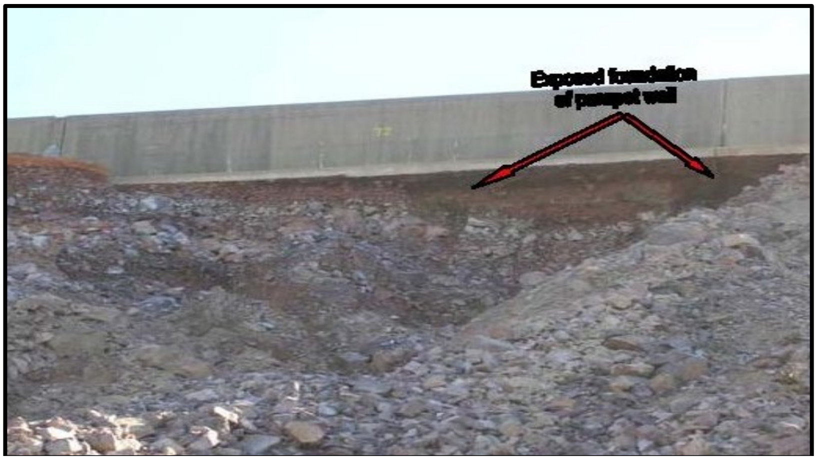

Figure 3: This photograph taken below panel 71- 72 shows the deep plunge pool that developed and subsequently undercutting of the parapet wall [3].

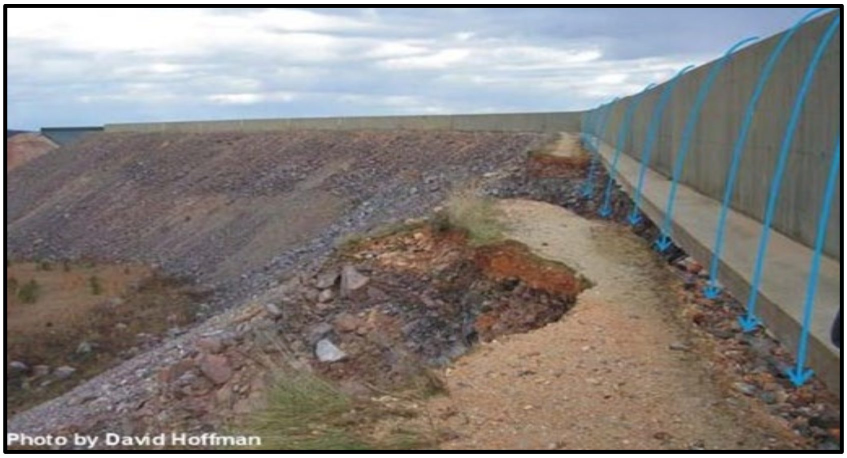
Photo by David Hoffman

Figure 4: photo shows arrows representing the outflow nape once it extended beyond the wall footing and directly onto the underlying rockfill. The rate of scour and erosion increased dramatically once this occurred [3].

The sequence of events from overtopping of the parapet wall until wall section collapses is explained in Figure (5).The rockfill dam was washed away while the failed section of the parapet wall overturned and collapsed. The progress of erosion of the dam embankment until the development of the breach is illustrated in enlarged diagrams as shown in Figure(6).The remnants of the lip as seen at the site of failure is shown in a photograph of Figure (7).

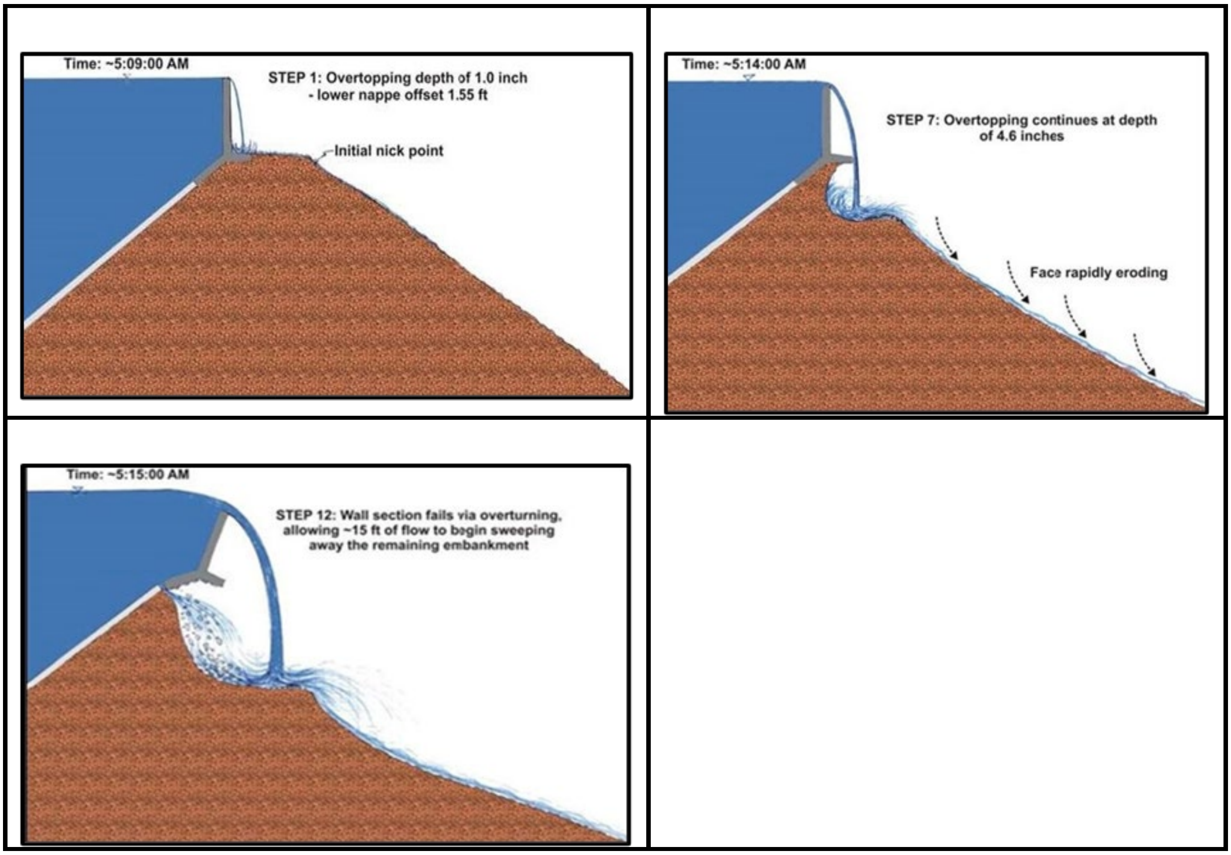
Time: ~5:09:00 AM
STEP 1: Overtopping depth of 1.0 inch - lower nappe offset 1.55 ft
Initial nick point
Time: ~5:14:00 AM
STEP 7: Overtopping continues at depth of 4.6 inches
Face rapidly eroding
Time: ~5:15:00 AM
STEP 12: Wall section fails via overturning, allowing ~15 ft of flow to begin sweeping away the remaining embankment

Figure 5: The sequence of event from overtopping the parapet wall until wall section collapses [3].

After failure of the dam, several investigation reports were prepared in an effort to identify the causes of the failure. The failure scenario was a case of overtopping of one section of the parapet wall on top of the reservoir dyke crest resulting in its collapse and leading to the release of water, which eroded the downstream slope of the dyke. Erosion was due to the rapid rise in phreatic surface and pore pressure in the dyke's fill. The collapse of the parapet wall section itself had happened when overtopping of the parapet wall had locally undermined its footing leading to its sliding and overturning. The reports agreed that; the water level monitoring instrumentation was wrongly set so that their sensors gave a lower level of water than real, and with the absence of visual confirmation of water level and lack of an emergency spillway, were all primary contributing causes to the failure. The same reports also identified the weak foundation conditions of the rockfill dyke and its low shearing strength together with the operation and maintenance of the dam itself were all secondary causes. The failure of the parapet wall which initiated the failure processes was taken as a tertiary contributing cause.

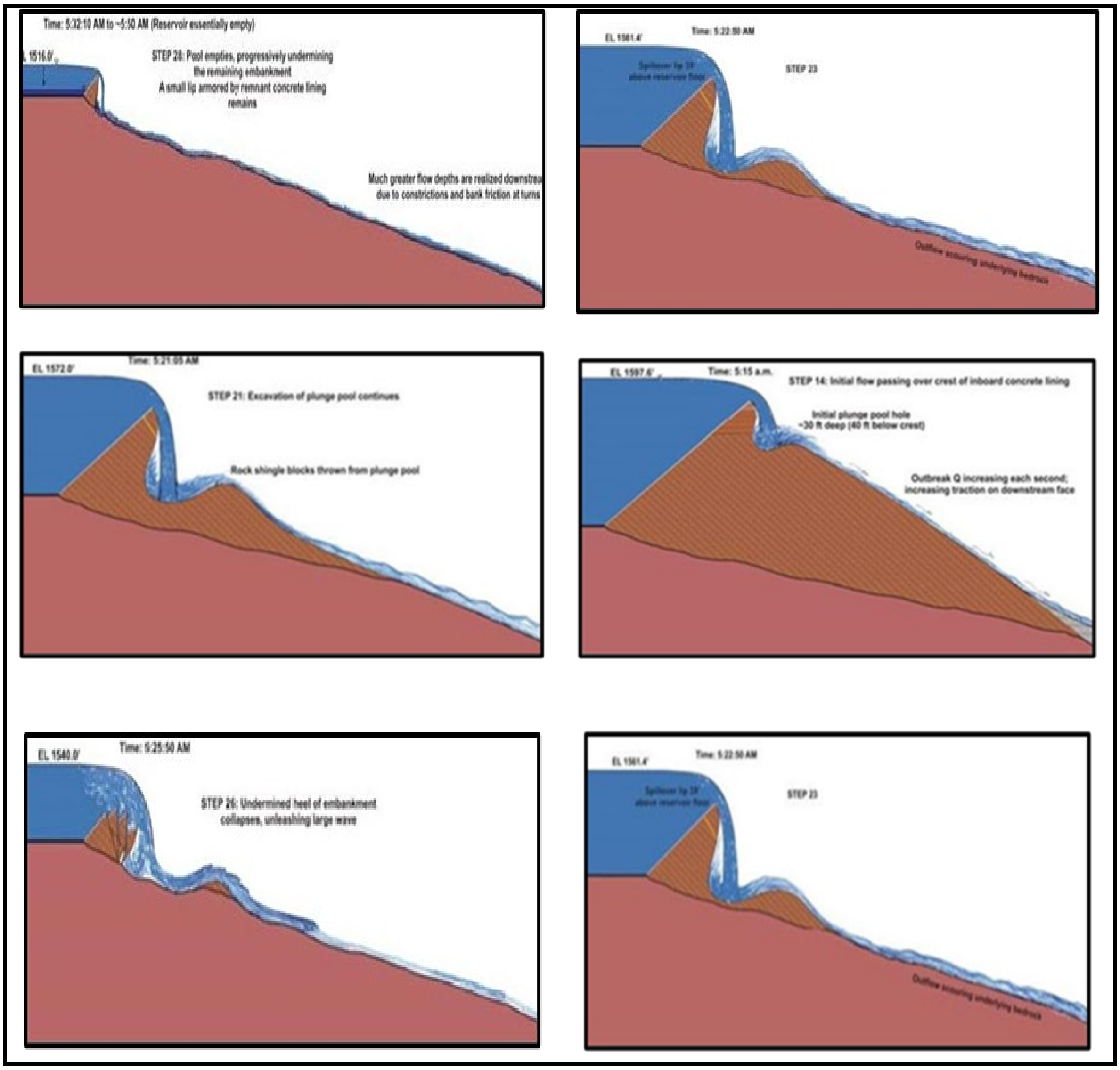
Time: 5:32:10 AM to ~5:50 AM (Reservoir essentially empty)
L 1516.0'
STEP 28: Pool empties, progressively undermining the remaining embankment
A small lip armored by remnant concrete lining remains
Much greater flow depths are realized downstrea
due to constrictions and bank friction at turns
EL 1561.4'
Time: 5:22:50 AM
Spillover lip 39' above reservoir floor
STEP 23
Outflow scouring underlying bedrock
EL 1572.0'
Time: 5:21:05 AM
STEP 21: Excavation of plunge pool continues
Rock shingle blocks thrown from plunge pool
EL 1597.6'
Time: 5:15 a.m.
STEP 14: Initial flow passing over crest of inboard concrete lining
Initial plunge pool hole ~30 ft deep (40 ft below crest)
Outbreak Q increasing each second; increasing traction on downstream face
EL 1540.0'
Time: 5:25:50 AM
STEP 26: Undermined heel of embankment collapses, unleashing large wave
EL 1561.4'
Time: 5:22:50 AM
Spillover lip 39' above reservoir floor
STEP 23
Outflow scouring underlying bedrock

Figure 6: Progress of erosion of the dam embankment until the development of the breach [3].

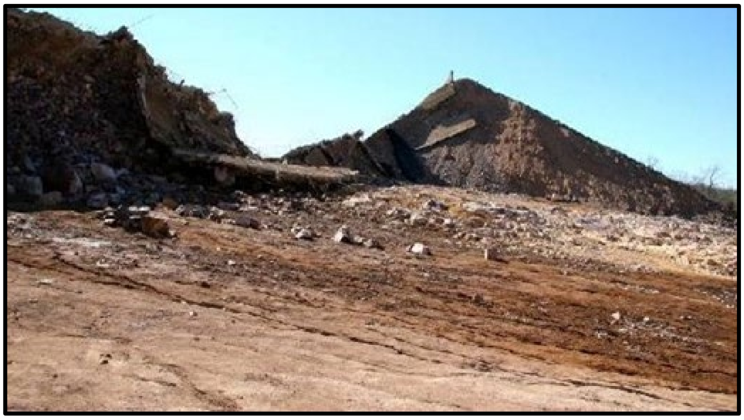

Figure 7: Photograph showing remnants of the lip as seen at the site of failure [3].

Summarizing, post-breach inspections and evaluations revealed the following information timeline:

1. The project historically operated with a minimum of two feet of freeboard on the lowest section of the parapet wall. Following installation of a geomembrane liner in 2004, the owner operated the project to fill the upper reservoir within one foot of the lowest section of the parapet wall. Post breach evidence shows the reservoir may have been routinely filled to within 0.25 foot of the lowest section of the parapet wall.
2. The December 14, 2005 breach was preceded by a significant wave overtopping that occurred on September 25, 2005.
3. On September 27, 2005, the owner's personnel adjusted the reservoir control programming to account for the difference between the actual reservoir levels and the readings from the reservoir level instrumentation as such difference was visually observed.
4. On October 3-4, 2005, the owner's personnel discovered that the conduit which housed the instrumentation for monitoring reservoir levels was not properly secured to the dam. Deterioration of the instrumentation tie-down allowed the conduits to move adversely impacting the reservoir level readings instrument. The instrumentation readings showed reservoir levels that were lower than actual levels. As a safety measure, the owner's personnel adjusted the reservoir level control programming to shut down the pumps when the instruments showed the reservoir levels were two feet lower than normal settings. Figure(8) shows the deflected conduits of the cables and the foot note below it explains the sequence of events leading to this event.
5. Two new conductivity sensors were installed as a safety system for shutting down the units in case of high water levels. The sensors would send a signal to shut down the units when they became wet. The sensors were mistakenly relocated to a height that was higher than the lowest point on the parapet

wall. Therefore, if the new sensors were contacted by water, the Upper Dam would already be in an "overtopping" condition.

6. Modifications made to the reservoir control programming adversely affected how the signals from the new sensors were managed and reported. The modifications required that both sensors contact water to initiate shutdown. This removed a layer of redundancy to the safety system [14] and [15].

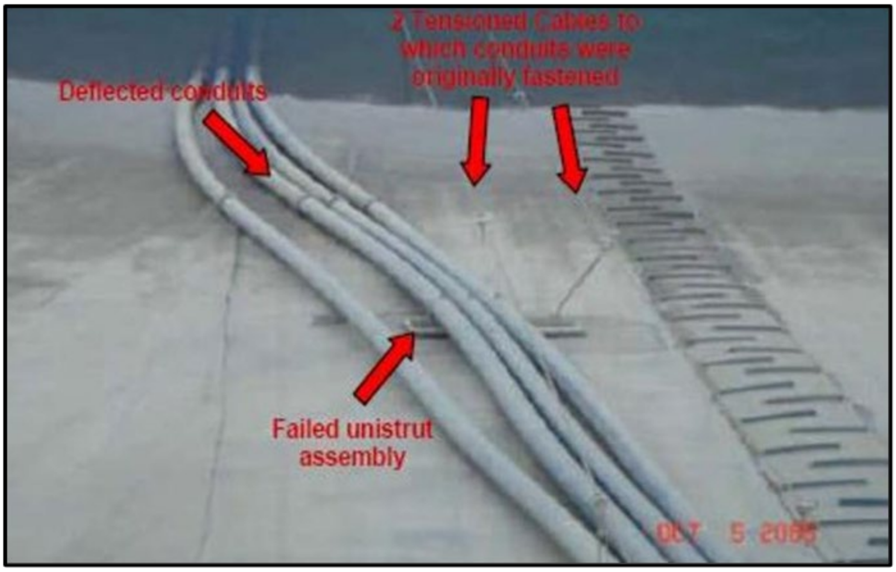
2 Tensioned Cables to which conduits were originally fastened
Deflected conduits
Failed unistrut assembly
OCT 5 2005

Figure 8. Deflected conduits at Taum Sauk Dam reservoir.

Note: Deflection of the instruments cables conduits was caused by the swirling action of water in filling/ emptying of the reservoir. This was due to the proximity of the conduits to the inlet/ outlet water shaft of the power station. This deflection was created after replacing the concrete lining by geomembrane and the failure of the tie down system to secure the conduits in its right position.

Now looking at this case as an unfortunate chain of actions, reactions and interactions, it is clear that many nodes on this chain were pushing towards the final outcome. Decisions made, and other ones not taken had sealed the fate of the project since the beginning and throughout its life. Selecting the location of the upper reservoir, design and implementation of the reservoir dyke and its parapet wall, replacing the concrete lining by geomembrane and choice of the tie down method of conduits, and the choice of the instrument cables conduits location close to the inlet/ outlet of the power station shaft are all examples of unfortunate decisions. Missed decisions which could have improved safety conditions would have been:

- If control feedback systems were provided,
- If an emergency spillway was constructed,
- If larger free board allowing for higher dyke settlement was considered;
- If a decision was made in the design stage to locate the water level measuring instruments away from the location of the inlet/ outlet water shaft

so would not have subjected the instrument cables conduits to the vortices and swirling of water going in and out of the shaft causing its shifting position leading and the instrument to indicate erroneous safe freeboard and therefor delaying shutting down of the pumping unit causing overtopping.

The final act which to say, the straw that broke the back of the camel, was the one dictated by society demanding more power at time of intense competition between utilities, which pushed the owners to delay repairing the water level measuring instruments until the normal outage for general maintenance.

The intense competition came as a result of deregulation of the electric market which meant that utilities were no longer guaranteed a rate of return on investment. Rather, in the emerging free market, utility profits were driven by market conditions. In December, 2005, the Taum Sauk pumped storage project provided significant financial benefits to its owner. For this reason, repairs were delayed until the planned future shutdown. In such case, safety was compromised by the competing goals of profitability and reliability as the repairs were delayed.

In final judgment, it should be noted here that the control equipment did not fail. The shutdown system behaved exactly as programmed but, unfortunately, the programing of the water level measurement was erroneously set, and this was a grave human failure. Moreover, even the erroneous readings of these equipment's were not a sufficient reason for the failure by themselves, but when compounded with other human mistakes and failings led to failure.

Construction of a new Upper Reservoir for the Taum Sauk Pump Storage Plant took place from 2007 to 2010. Today the reservoir is impounded by a roller-compacted concrete dam that is equipped with a multitude of safety features and appurtenances that adhere to current standards [16].

4. The Extraordinary Incidents and the Human Factor

4.1 General

The category of “Extraordinary Human Caused Incidents”, occur not as “Normal Accidents”, but result from thoughtful and carefully meditated decision making human process for the purpose of destabilizing dams or any other strategic or economic system. These have been manifested in the long history of dams in acts of war or sabotage, and in modern history by cyber-attacks which aim at disrupting the normal operation of dam or system of dams, leading to substantial deviation from the operational state as per design intents. The final objective is creating an unacceptable risky condition and damage.

History tells that using water released from dams or canals was an old used war tactic to destroy enemy troops, or prevent their advance, or even facilitate own army advance. Cyrus the Great reputedly took Babylon in a single night in the 6th century, BC., by diverting an old artificial lake back into the Euphrates, so that his army could come right up to the city walls at night.

Hulagu, who destroyed medieval Baghdad in 1258 AD. used the Tigris River flood waters to trap the caliph's horsemen outside the city walls; and, the Mongols also destroyed the medieval city of Gurjang in Central Asia by breaching a nearby dam, making it an example of those who dared resist their advance.

In the 1980s, both Iran and Iraq used water as an area denial weapon to check the other's advance in southern areas of both countries. Iran tried to bomb Iraqi dams out of commission, and Iraq retaliated in the same way in the first Gulf War. USA and the Coalition air forces did exactly the same thing in the Second Gulf War to knock out hydropower stations such as that of Mosul Dam. These acts were all done while the 1977 Geneva Conventions specifically outlawed the targeting of water infrastructure in wartime.

Spectacular water warfare methods used in recent times are still vivid in the human memory. In the Second World War, the occupying Germans broke dikes in Netherland to try to halt the Allied advance. This did not delay the Allies much, but it did destroy about a quarter of the country's total farmland ahead of a very bitter winter. The Allies also blew up dikes in The Netherlands for tactical purposes, but not on a large scale. The Germans also flooded terrains in Italy in order to deny them to the Allies, leading to terrible malaria outbreaks. Adolf Hitler reserved the worst for his own Reich; however, his scorched-earth "Nero Decree" would have destroyed the German hydroelectric and flood control systems on the cusp of German defeat in 1945; if it were not for wise German officials who declined to carry out the suicidal order.

The British, being on the verge of defeat by the Germans, did not hesitate from committing the same type of atrocities when the RAF carried out the famous dam-busting action in Europe in World War II. The flooding from the breaches killed more than a thousand German civilians plus many Allied prisoners trapped in downstream camps.

In one case, the sheer loss of life of one area-denial far surpassed any chastise. Hundreds of thousands of Chinese civilians died when the Nationalist Chinese breached the Yellow River dikes in June 1938. Nationalist generals planned to "use water as a substitute for soldiers" during the battle of Wuhan, which proved to be a hasty decision. Nationalist soldiers bombed and hacked at the dikes for days until the first breach took place on June 9. There was no coordinated evacuation for the people in the water's path, nor even many early warnings. Most officials had already fled ahead of the Japanese army and very few households had radios or telephones. Neither the retreating nationalists nor the Japanese occupiers provided much relief to the survivors. And no civilian aid organizations could get into the disaster zone due to the fighting. Owing to the flooding, the Japanese army had to give up on its immediate target, the city of Zhengzhou. The floods failed to immediately halt the main Japanese offensive on Wuhan, but the city fell in October 1938.The Nationalist government tried to blame the disaster on the Japanese even though refugees, the military, and the foreign press all knew quite well that Chinese spades and mortars were responsible [17]. The increased use in modern times of dams and water structures as targets by terrorist groups to achieve political ends has undermined the safety of those infrastructures. This has led in the meantime to increased governments worries and increased dam hazards.

In the recent technological and scientific revolution, new threats from the "Human Factors" have resulted from the interference with modern technologies by wrong manipulation, which can destabilize and hit systems used in dams.

A recent report prepared by the Office of the Inspection General of the US Bureau of Reclamation under the title "External direct impacts- Selected Hydropower Dams at increased Risk from Insider Threats" presented this by explaining that the world is witnessing an ever expanding revolution in the networked information infrastructure that blends computing, and communications and this may be considered as the greatest achievement in human history. So, during the last two decades advances in the information and communication technology (ICT) have fundamentally reshaped the management policies and control procedures of complex systems all over the globe making great savings in costs and increasing flexibility in operation, but the report adds that this has increased at the same time the threats from human factors.

In the two dams examined by the Bureau, "Direct Threats" on the Bureaus' dams were considered minimal since the operation (ICT) systems were isolated and independent from the internet and from the USBR's business systems. The report, however, warns from the "Indirect Threats" coming from accounts management and personnel security practices, which puts the control ICS and the infrastructure they manage at high risk from insiders' threats.

Typically, loop holes can develop from failure to limit the number of ICS users who have administrator access and having an extensive number of group accounts, which can allow "Hackers" creeping in together with the noncompliance with password policies and failure to remove inactive system administrator accounts.

The report also warns against not following best security practices so it recommends that personnel with elevated system privileges complete more rigorous background investigations. Deficiencies, in the words of the report, have occurred because USBR management failed to strengthen the Bureau Risk Management Practices in response to rapidly escalating threats of modern warfare and cyber-attacks. The report even went further to cite spectacular examples from recent years, including targeting electric power generators and distributors in the Ukraine by infecting control systems that operate the infrastructure. This was linked to Russia, which was accused of using sophisticated malware [18].

Recognizing the grave consequences, it was in 2016, that for the first time, the Industrial Control Systems Cyber Emergency Response Team in the US (ICS-CERT) included dams in its assessments along with other types of infrastructure such as chemical plants, manufacturing facilities, and wastewater treatment. According to one report, ICS-CERT had performed 98 assessments and recorded 94 instances of weak boundary protection of the control system which could facilitate unauthorized access. There were also incidences of unnecessary services, devices, and ports on control systems, as well as weak identification and authentication management. Furthermore, large and significant dams were considered to be at risk from unauthorized access.

4.2 Human Factors in an Example of Cyber Threats to Compromise Dam Operation

Cyber threats are viewed as a growing concern in the dams' community due to implications for public safety. Increasing vulnerability has been created due to facilities' previously manually operated components becoming more complex and supplemented with remote capabilities. As the number of connected technologies in a facility's control systems, such as in dams' increases, so does the cyberattack exposure of those systems. Automation has its benefits, such as efficiency and capturing real-time data and information, but it does also create new risks.

Opening the Flood Gates of a dam in the wrong time can lead to catastrophe. If this operation is carried out by a hacker; then such thing becomes reality more than fiction. In the scenarios developed for such occurrences, one hacker seeks to create significant disruption in opening of the flood gates at a dam. If any such scenario were to occur, it is likely to cause significant downstream flood damages in addition to public outcry. It is only natural then that the cyber security of important systems has gained lately special weight.

According to the report "Silent Cyber Scenario: Opening the Flood Gates", the cyber security of critical infrastructure, such as dams, has become a focal point in recent years. The report cites the event in 2013 when the control system at Bowman Avenue Dam in the United States was breached for about three weeks. The hacker obtained access to remote operation of the dam gates, which had fortunately been taken offline for maintenance [19].

Although the Bowman Avenue Dam is a small dam (Figure 9), the manipulation of its gates by one hacker raised high concerns of such a possibility in lager dams. Hackers traced to Iran infiltrated the control system of this dam located just about 30 miles north of Manhattan; in 2013. The floodgate of the Bowman Avenue Dam is just 15 feet wide and two and a half feet tall, but cybersecurity experts say if the Iranians were able to access its control system, then they could likely get inside systems for more significant infrastructures, such as pipelines, mass transit systems and power. The incident itself passed without causing any damage because the structure was in "maintenance mode ". But, if the hacker had been able to open the floodgate during a storm, then this could have caused nearby homes and businesses to flood.

The news of this incident brough the dam into the spotlights. In the words of Manhattan U.S attorney; the infiltration of the Bowman Avenue Dam represented a frightening new frontier in cybercrime. He also added, "we now live in a world where devastating attacks on our financial systems, our infrastructures and our way of life can be launched from anywhere in the world, with a click of a mouse".

Cyber security expert Joe Weiss commented on the incident by saying, "even if local flooding is the worst that could have happened if the hacker opened the floodgate, the incident shows how vulnerable infrastructures are to such threats", adding ,"the control system for the Bowman Avenue Dam is likely similar to those for more significant structures and that the same identical problems can happen in power plants, refineries, pipelines, transportation and even in nuclear plants".

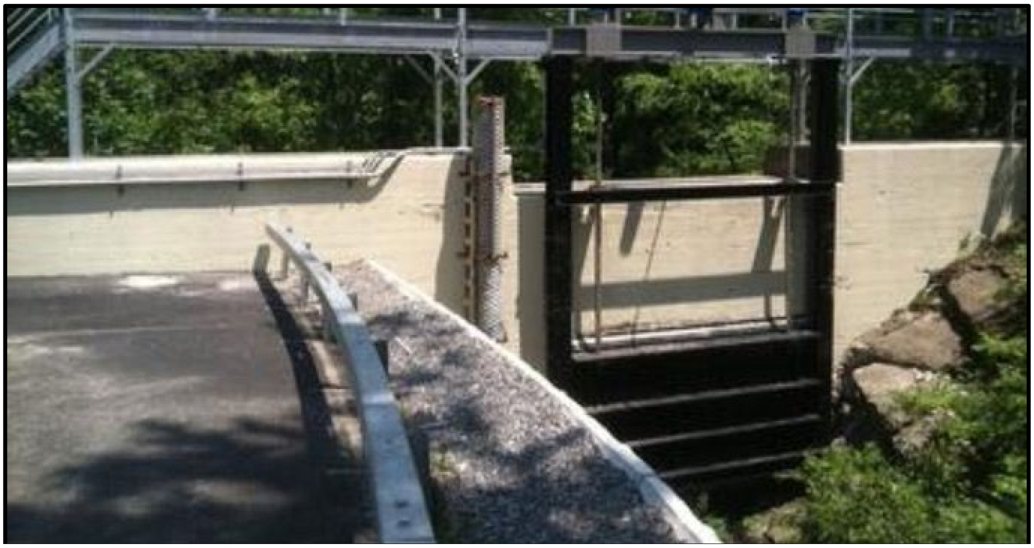

Figure 9: Downstream view of the Bowman Avenue Dam.

According to Weiss who maintained a database of "cyber incidents" involving control systems dating to the 1980s. Though not all of the 800 incidents were "malicious," he says, they have led to some 1,000 deaths. Ten of the overall incidents involved dams. He mentions that it is often that hydro facilities are in the middle of nowhere, and they are in many instances are unmanned, and so there is need for some sort of remote monitoring and remote control to avoid major problems. One such problem occurred at the Taum Sauk Hydroelectric Power Station in Missouri in 2005 which was described in this paper, when the failure of water-level gauges is believed to have caused water to overflow and part of a reservoir to collapse, which injured several people. The incident was the result of a control system and human multi failures, not a hacking or cyber-attack, but Weiss says: "It could have been done maliciously, and very easily" [20] and [21].

The question of incursion on dams' safety by cyber means, whether to be considered as an armed attack threshold at which a State may take forceful action in self-defense or not, has received considerable study by International Law experts. In general, if the consequences of cyber operations targeting a State's infrastructure are destructive and injurious; then it is more likely that such cyber-attacks be considered the same as an armed attack. The approach of the Tallinn Manual on the International Law applicable to Cyber Warfare which was formulated between 2009 and 2012, considers that cyber actions that qualify as an armed attack open the door to a forceful response, by either cyber or non-cyber means, pursuant to the law of self-defense, though this must be assessed on a case-by-case basis.

The US views is precisely the same. Harold Koh, the then-legal adviser at the US State Department, explained in 2012 that “Cyber activities that proximately result in death, injury, or significant destruction would likely be viewed as a use of force.” He noted that “commonly cited examples of cyber activity that would constitute a use of force include, for example, (i) operations that trigger a nuclear plant meltdown; (ii) operations that open a dam above a populated area causing destruction; or (iii) operations that disable air traffic control resulting in airplane crashes”. Koh stated that “when ~~“~~assessing whether an event constituted a use of force in or through cyberspace, we must evaluate factors: including the context of the event, the actor perpetrating the action (recognizing challenging issues of attribution in cyberspace), the target and location, effects and intent, among other possible issues [22].

4.3 Human Factors and the Hazards to Dams Created From Acts of War

Destruction of dams during times of war is one more source of concern to governments; this concern is reasonably justified since destruction of dams wasbeen done during wars throughout history. Such destruction, however, took much greater dimensions in World War II with the objectives of inflicting maximum human losses in the civilian population, and paralyzing warring parties by disrupting the industrial production supporting the enemy’s war effort. In the following, some cases are described when dams were the subject of war hostilities.

4.3.1 Bombing of the German Dams in World War II

In this case bombing raids were carried out by the British Royal Air Force (RAF) to destroy three dams in the Ruhr valley in Germany on the night of 16-17 May 1943 in what was called operation chastise. The dams were fiercely protected. Torpedo nets in the water stopped underwater attacks and anti-aircraft guns defended them against enemy bombers; these dams secured the water supply for much of the surrounding areas and industries. It was planned that the destruction of these dams in this region would cause massive disruption to the German war effort. The plan, however, required the development of a weapon capable of destroying these dams and converting the Lancaster type bomber aircraft to deliver it. The bomb was needed to be dropped from a height of 18m at a ground speed of 232 mph (374 kph). The bomb would spin forwards across the surface of the water before reaching the dam. Its residual spin would then drive the bomb down the wall of the dam before exploding at its base, Figures (10 and 11). The three main targets were the Möhne, Eder and Sorpe dams. The Möhne dam was a curved “gravity” 40 m high and 650 m long concrete dam, Figure (12). There were tree-covered hills around the reservoir, but any attacking aircraft would be exposed on the immediate approach. The Eder dam was of similar construction but was an even more challenging target. Its winding reservoir was bordered by steep hills. The only way to approach would be from the north. The Sorpe dam was a different type of dam and had a watertight concrete core 10 m wide. At each end of its reservoir, the land rose steeply making the approach path of the attacking aircraft exceedingly difficult.

The Möhne dam was attacked first at 12.28 am and was breached, Figure (13), then the Eder dam was next which in its turn collapsed at 1.52 am. Meanwhile, aircrafts from the two other waves bombed the Sorpe dam, but it remained intact.

The two direct bombs hits on the Möhne dam resulted in a breach around 76 m wide and 89 m deep. The destroyed dam poured around 330 million tons of water into the western Ruhr region. A torrent of water around 10 m high and travelling at around 24 km/h swept through the valleys of the Möhne and Ruhr rivers. A few mines were flooded; 11 small factories and 92 houses were destroyed, and 114 factories and 971 houses were damaged. The floods washed away about 25 roads, railways and bridges as the flood waters spread for around 80 km from the source. Estimates show that before 15 May 1943 steel production on the Ruhr was 1 million tons; this dropped to a quarter of that level after the raid.

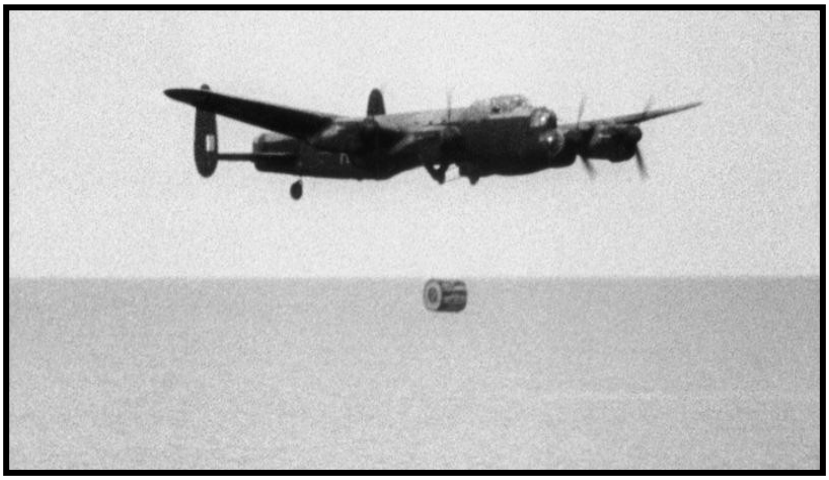

Figure 10: View of the Lancaster bomber and the bomb to attack the German Dams [23].

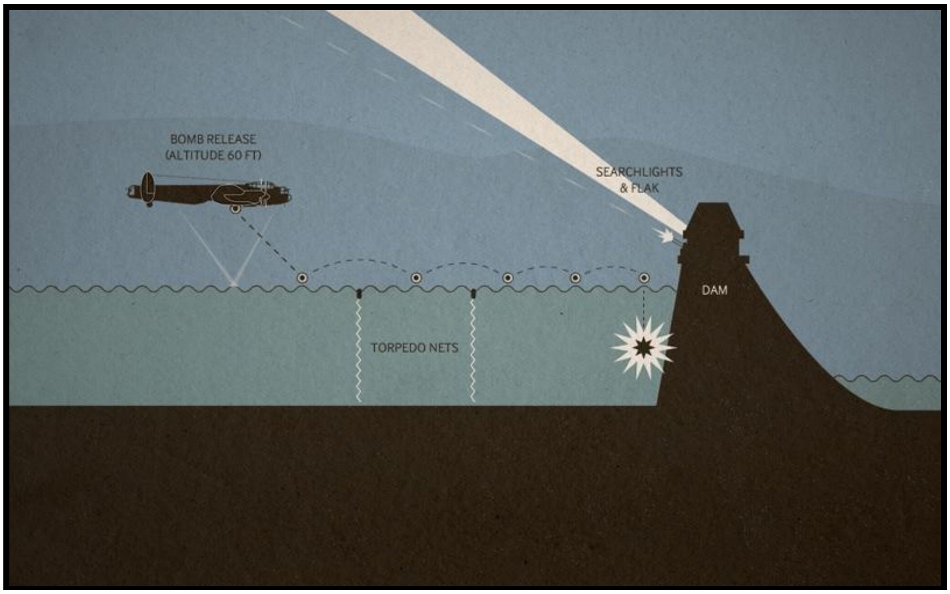

Figure 11: The visualized attack by the bouncing bomb on the German Dams [23].

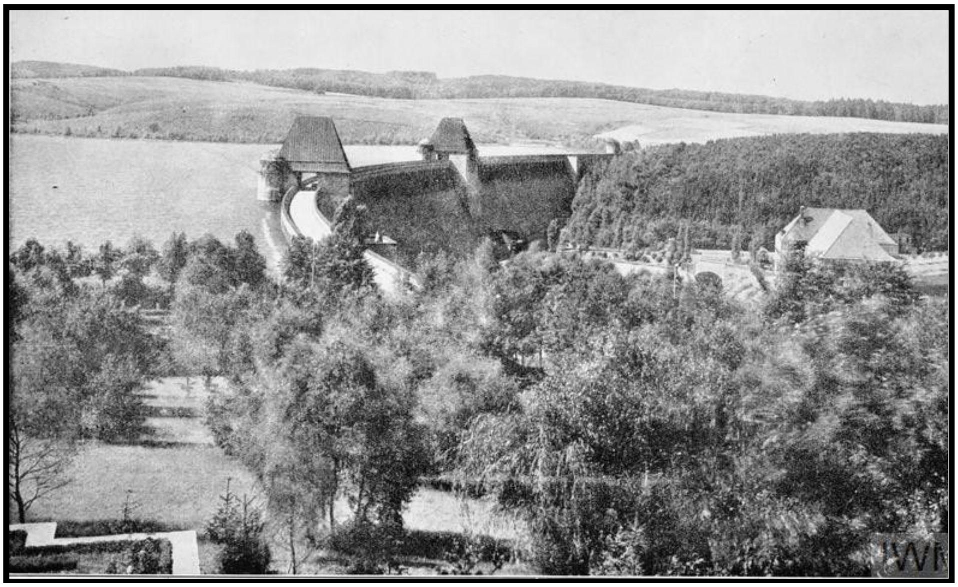

Figure12: View of the Möhne dam [23].

In the case of Eder Dam, the main purpose of which was to act as a reservoir to keep the Weser and the Mittelland canal navigable during the summer months, the wave from the breach was not strong enough to result in significant damage by the time it hit the city of Kassel approximately 35 km downstream.

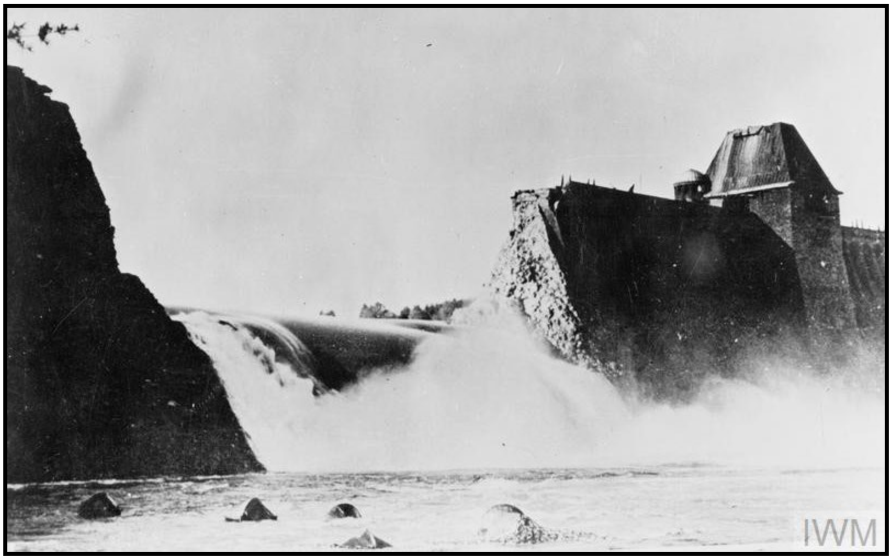

Figure13: View of the Möhne dam breaching [23].

The greatest impact on the Ruhr armaments production was the loss of hydroelectric power. Two power stations, producing 5,100 kilowatts, associated with Möhne dam were destroyed and seven others were damaged. This resulted in a loss of electrical power in the factories and many households in the region for two weeks. In May 1943, coal production dropped by 400,000 tons which German sources attribute to the effects of the raid [24].

Although the raids were considered successful at the time many of the attacking airplanes were hit and crashed. Of the 133 aircrew that took part, 53 men were killed and three became prisoners of war.

On the ground, almost 1,300 people were killed in the resulting flooding, but the impact on industrial production was limited [23].

The principle of the dam buster bomb was described by its inventor Barnes Wallis in his 1942 paper “Spherical Bomb”, as an attack in which a weapon would be bounced across water until it struck its target, then sink to explode underwater, much like a depth charge. Bouncing it across the surface would allow it to be aimed directly at its target while avoiding underwater defenses, as well as, some above the surface, and such a weapon would take advantage of the "bubble pulse" effect typical of underwater explosions, greatly increasing its effectiveness, Figure (14).

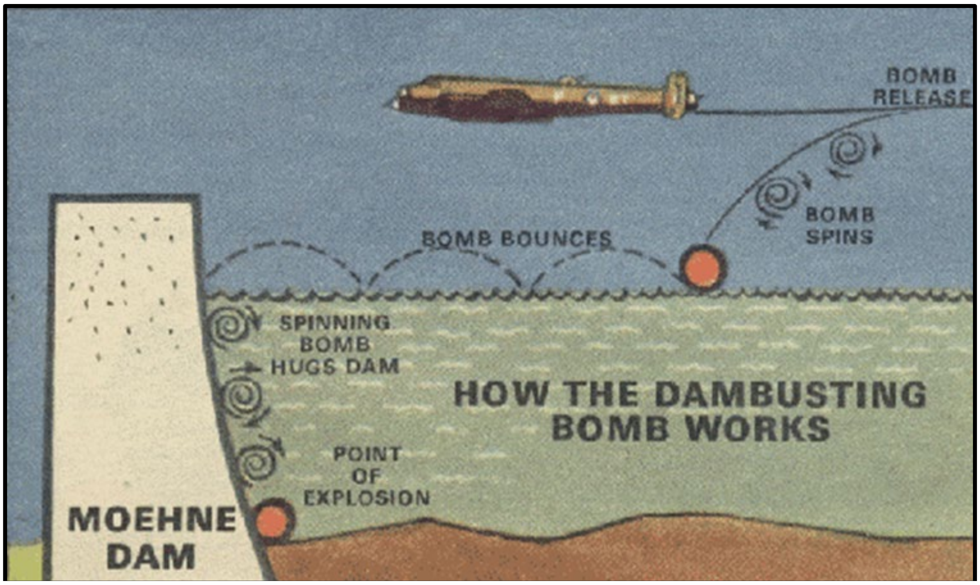

Figure 14: Bouncing bomb principle [25].

Wallis's paper identified suitable targets as hydro-electric dams and floating vessels moored in calm waters. The principle on which the bomb works were based on creating an earthquake impact on the on the dam it hits [26].

4.3.2 Blasting the Dnjeprostroj Dam

Another instance of targeting dams in wars is the blowing up of Dnieper Hydroelectric Station in the Ukraine: also known as Dnjeprostroj Dam; which is the largest hydroelectric power station on the Dnieper River. The station was built in two stages. Dnipro- HES-1 was originally built during 1927-32, but it was destroyed during World War II to make use of the river as a natural obstacle.

The strategically important dam and plant was dynamited by retreating Russian Army troops in 1941 after Germany's invasion of the Soviet Union. An account which described the aftermath of the Russians action went into details of how the explosion had flooded villages and settlements along the Dnieper River.

The tidal surge killed thousands of unsuspecting civilians, as well as Red Army officers who were crossing over the river. Then, it was partially dynamited again by retreating German troops in 1943. In the end, the dam suffered extensive damage, and the powerhouse hall was nearly destroyed. Both dam and station were rebuilt between 1944 and 1949. The Dnepr- HES-2 was built in 1969-80, which during the 2000s was modernized. Figure (15) shows the original dam under construction, and Figure (16) shows it after being damaged, while Figure (17) shows it after modernization [27].

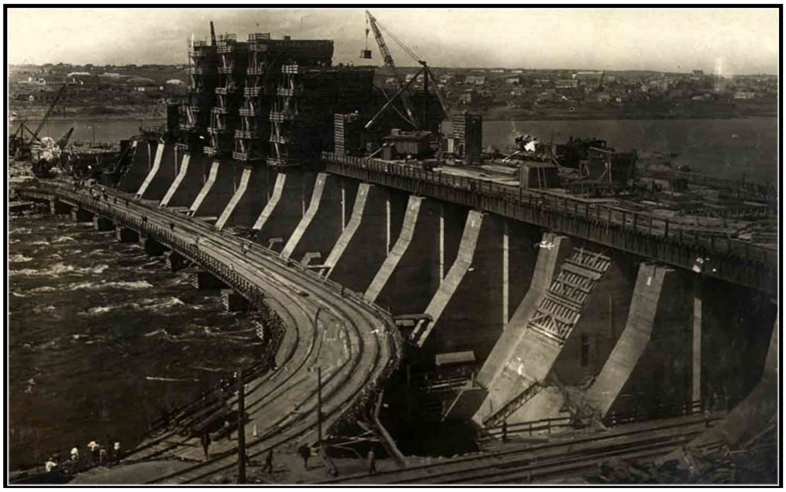

Figure 15: The dam under construction [27]

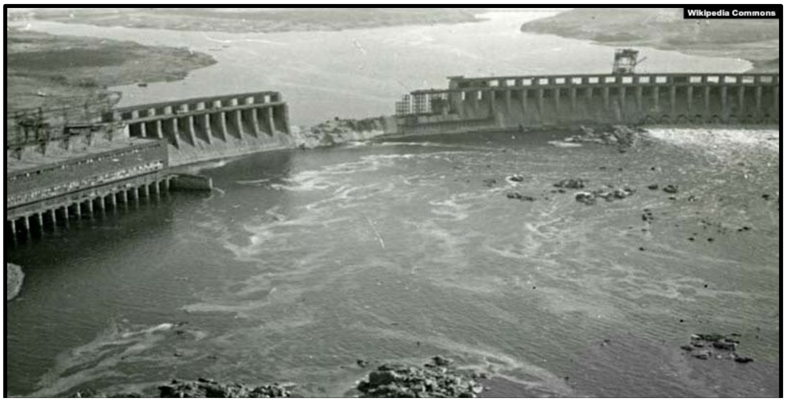

Figure 16: The dam after the incident [28]

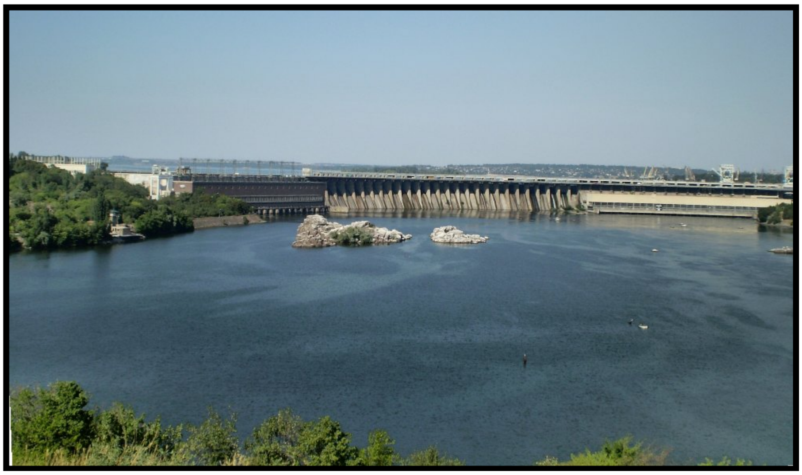

Figure 17: The dam after modernization [27].

Since no official death toll was released at the time, the estimated number of victims varies widely. Most historians put it between 20,000 and 100,000, based on the number of people then living in the flooded areas [28].

4.3.3 Burguillo and Ordunte Dams Attacks

Burguillo and Ordunte concrete gravity dams were attacked and damaged by the Nationalist army in the Spanish civil war in 1937.

The Burguillo Dam near Avila, Figure (18) [29] is 91m high, which has a reservoir of 287.22 million cubic meter capacity. The second one is 56 m high Ordunte near Bilbao, Spain.

It was reported that a 2.5ton charge was detonated in an inspection gallery at Ordunte. Some limited damage resulted. The dams were repaired in 1938 – 1939 [30].

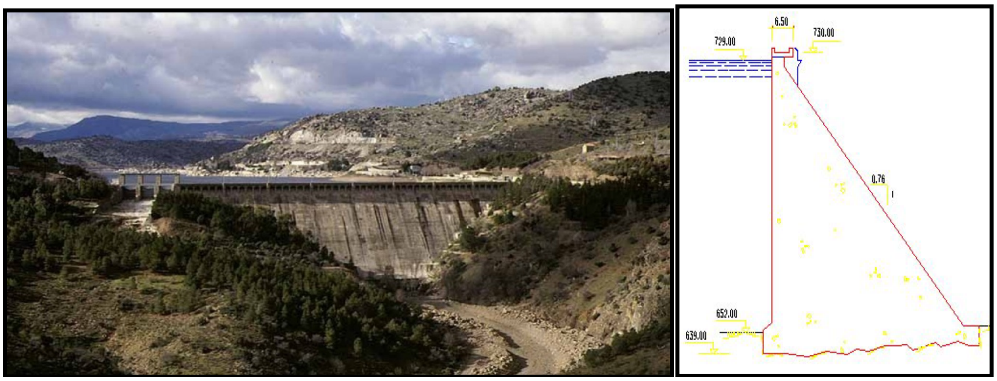

Figure18: View and cross section of Burguillo Concrete Gravity Dam completed in 1913 [30].

4.3.4. Hwacheon Dam Attacks

From the Korean War, the example of the Hwacheon dam can be cited. This is a concrete gravity dam 81.5 m high, 435 m long and holds a reservoir of 1,018 million cubic meters of water. It is located on the North Han (Pukhan) River in Hwacheon County, Gangwon-do Province, South Korea. The dam was completed in 1944 as a primary source of electricity in southern Korea, and it also provides flood protection from North Korea's Imnam Dam upstream. It was the focal point of a raid during the Korean War. At midnight 8 April 1951, North Korea and Chinese forces released excess water from the dam's spillway which disabled five floating bridges of the United Nations Command downstream. On 9 April, the American forces were charged with the task of capturing the dam but failed. Between 16 and 21 April, Allies had secured the dam but were repelled by Chinese counterattack before being able to destroy the dam's floodgates. After an unsuccessful attempt to destroy the dam using heavy bombers, another raid was more successful on the 1st of May. The American use of aerial torpedoes was successful by dropping 2,000-pound torpedo bombs on the dam, puncturing one spillway gate, Figure (19) [31].

On 1st May 1951, US Air Group 19 assaulted the dam with eight Skyraiders attack bombers that were equipped with Mk 13 torpedoes and escorted by twelve corsairs fighter planes. Seven of eight torpedoes of 2,000 pound bombs struck the dam, and six exploded. The attack alleviated the dam as a flood threat, destroying one sluice gate and damaging several others [32]. This raid constitutes the last time globally that an aerial torpedo was used against a surface target, and was the only time torpedoes were used in the Korean War.

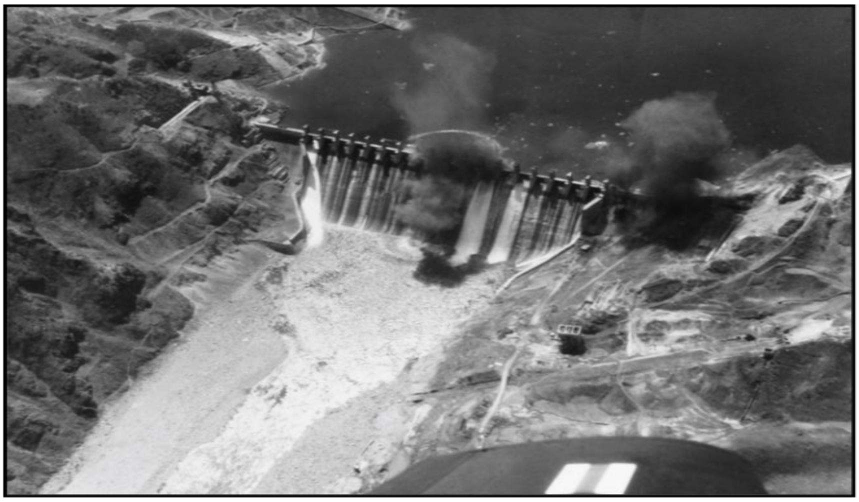

Figure 19: Hwacheon Dam being hit in the Korean War in 1951 [31] and [32].

4.3.5 Peruća Dam

The blasting of the 63 m high earthfill Peruća Dam in Croatia in 1993 is the last example of these illustrations. The dam was constructed on Cetina River in 1958. The dam's reservoir active storage is 565 million cubic meters at the maximum operating water level at elevation 361.50 m (a.s.l.). The maximum reservoir flood level is 362.00 m (a.s.l.). The dam affects the Cetina flow regulation at the downstream power plants between Sinjsko Polje and the Adriatic considerably.

The Peruća Dam was greatly damaged during the Croatian War of Independence, when on January 28, 1993 the dam was blown up by Serbian/Yugoslav army forces. They mined it with 30 tons of explosives and detonated the charges with the intention of harming thousands of Croatian civilians downstream. The explosion caused heavy damage, but ultimately failed to demolish the dam. The Croatian communities in the Cetina valley were, nevertheless, in great danger of being flooded by water from Peruća Lake. The actions of an officer from the United Nations Protection Force (UNPROFOR) which was the first United Nations peacekeeping force in Croatia, Bosnia and Herzegovina during the Yugoslav Wars prevented the disaster at the Dam. Before the explosion, raised the spillway's gates were raised and reduced the level of water in the lake by four meters. This prevented total collapse of the dam, and engineers were quickly able to maintain the integrity of the dam. Subsequently, the Croatian forces intervened and recovered the dam and the surrounding area. On January 29, 1993 a small Croatian army team, supported by engineers previously employed in dam maintenance could get access to the main outlet valve which was stuck due to two years of neglect. Loaded with 700 tons of hydrostatic pressure on the valve, the engineers managed to refill the oil in the hydraulic pumps and used an UNPROFOR engine to restart them. This allowed for the lake to finally drain into Cetina River at rate of $187m^3/sec$ [33] and [34].

4.4 Human Factors and the Hazards to Dams From Terrorists Actions

4.4.1. General

Terrorism is defined as the acts of violence intentionally perpetrated on civilian non-combatants with the goal of furthering some ideological, religious or political objective. In the current national security environment, there is little question that terrorism is among the gravest of threats [35].

The importance of freshwater and water infrastructure to human and ecosystem health and to the smooth functioning of a commercial and industrial economy makes water and water systems targets for terrorism. The chance that terrorists will strike at water systems is real; indeed, there is a long history of such attacks [36].

Water infrastructures such as dams and water purification plants can be targeted directly causing flooding, or water can be contaminated through the introduction of poison or disease-causing agents, and in both cases, this can cause mass killing of people. This hazard has been recently recognized as one of the deadliest threats to dams' safety when used as either targets or tools of violence or intimidation by non-state actors. From recent history ,some cases can be cited for using dams as a political tool. In one case reported by the ITAR-Tass News Agency; a threat was made on November 6th, 1998 by a guerrilla commander, Col. Makhmud Khudoberdyev, who threatened to blow up a dam in Tajikistan unless his demands were met. This rebel commander, in the northern part of the former Soviet Republic, threatened to blow up this particular dam and flood vast areas of Central Asia if the government did not meet his demands. "It will flood vast territories of Central Asia," Col.

Makhmud Khudoberdyev warned in a statement received by the ITAR-Tass news agency. He said his guerrillas had mined the dam on the Kairakkhum channel as a "deterrence measure." The dam's reservoir is large enough that it was referred to locally as a "sea". The government, which had no immediate reaction to the threat, said it had surrounded Khudoberdyev headquarters and recaptured Khodzhand, a city in the north of this small, impoverished nation.

Dozens of people were killed in the recent conflict there. The rebels had opposed the previous peace agreement ending the country's six-year-old civil war and wanted fresh elections [37].

In other case rebels from the Democratic Republic of Congo carried out attacks in 1998 on Inga Dam during efforts to topple President Kabila, disrupting electricity supplies from the dam and water supplies to Kinshasa, Congo (refer to case No. 194 in [38]). The rebels overran Inga hydroelectric dam in the early phase of their offensive on the capital Kinshasa from bases in the southwest of the country. Since that time, they repeatedly interrupted the power supply to the capital, leading to the disruption of running water supplies to the population. Medical services and supplies were also severely affected by the outage. Following the recent intervention on the government side of forces from Angola and Zimbabwe, the military balance dramatically shifted in favor of loyalist forces and their allies. According to reports in an Angolan government-owned newspaper, rebels besieged in the dam area threatened to destroy the electricity installations if they were not granted safe passage out of there [39].

To shed more light on this hazard two more cases are presented in the following;

4.4.2. The case of Zgorigrad and Vratsa dam:

In a case of sabotage which was suspected as a terrorist action was that of Zgorigrad and Vratsa mining dam in Bulgaria in 1966. This dam was an earthfill tailing dam with a puddle concrete membrane impounding a sediment basin for lead and zinc mine called Mir located uphill of the village of Zgorigrad near Vratsa.

The breaching and collapse of the dam created 4.6 m high flood wave through the towns of Zgorigrad and Vratsa. Reports indicated that as many as 600 people perished in this disaster [40], [41] and [42].

The catastrophe of Zgorigrad is one of the worst disasters caused by the failure of tailings dams worldwide. On 1st May 1966, the failure of this tailings dam gave rise to a 450 000 cubic meter of mud flow, which ran for a distance of 6 km as far as the town of Vratza and caused the loss of many hundreds of lives, as well as vast material and environmental destruction. The slurry and water contained poisonous chemical elements [43] and [44].

Quoting from another report in Bulgarian language, it stated the following;

"*Officially, the Communist authorities at that time announced only about 100 deaths, but later it became clear that the victims of the incident were over 500 (there are reports of 118 corpses unidentifiable, of which 4 children), injured 2000, more than 150 houses destroyed, 300 families are left without a home, and the roofs of 1000 houses are taken from the elements".*

The same report contains some photographs taken in the aftermath of the failure. Remains of the dam's outlets which take the water out from the sedimentation basin after the event is presented in Figure (20).

While Figure (21) shows the people of Zgorigrad cleaning the city streets from the metallic mud that covered everywhere, and Figure (21) illustrates the extent of ecological damage inflicted by the disaster on the downstream area and the official work to remove as much as possible of the mud [45].

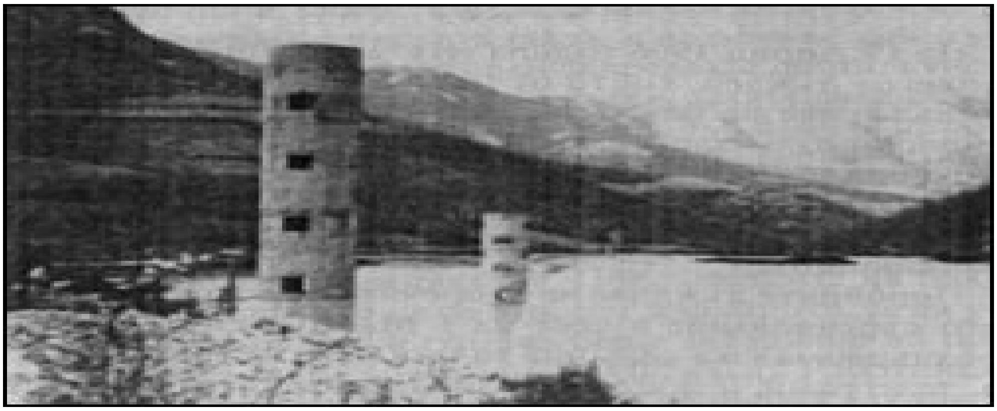

Figure 20: Photograph showing outlet spillways of the sedimentation basin [45].

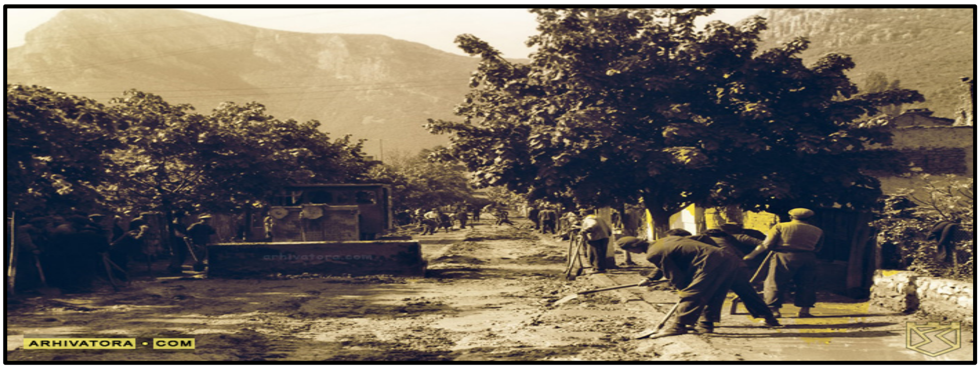

Figure 21: Cleaning of Zgorigrad streets from the toxic mud [45].

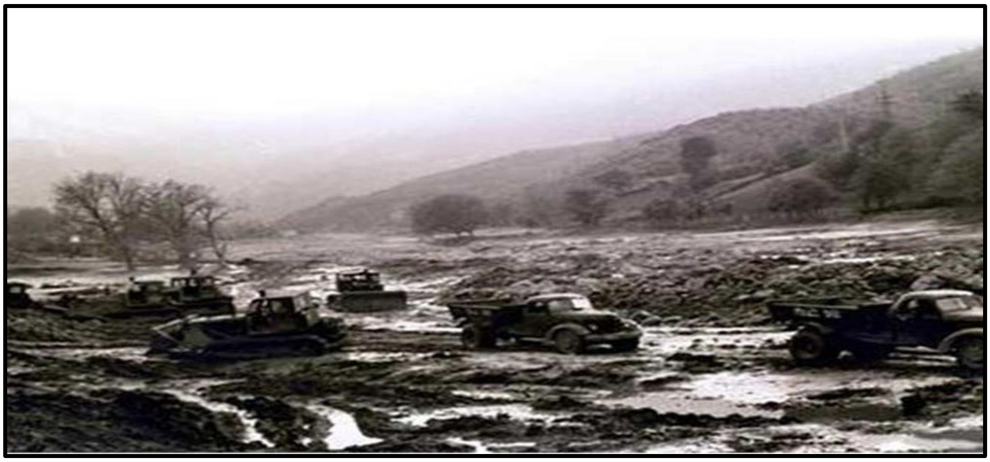

Figure 22: Cleaning operations in the flooded downstream Area [45].

4.4.3 The Rise of ISIS and the Threats it Presented to Dams in Iraq

The rise of the Islamic State in Iraq (ISIS), known also in the media as the Islamic state in Iraq and the Levant (ISIL), and its control of vast swathes of land in Iraq and Syria in 2014, can be considered as the most dramatic event in the recent history of the two countries and in the world. This fundamentalist fanatical religious group may be considered as the most radical group so far in modern history seeking to change the values and systems in the Islamic world. Therefore, establishing their brand of governance through the use of violence and targeting the population to force them into submission, and at the same time destroying most of the existing economic and social infrastructures in an attempt to weaken the authority of government(s).In their ways and methods, they used water as a weapon. One of the earliest examples was their seizure of Fallujah Barrage on the Euphrates River. The dam helps distribute water from the Euphrates River on its course through the western province of Anbar, and is located some 5 km south of Falluja town some 70 km (44 miles) west of Baghdad, the town which was overran by the militants early in their campaign. Early February 2014, ISIL took control of the Nuaimiya area where the dam is located, and began fortifying their positions with concrete blast walls and sand bags. Early in April of the same year, the militants closed eight of the dam's 10 gates flooding land upstream and reducing water levels in Iraq's southern provinces through which the Euphrates flows before emptying into the Gulf. In the following week, the militants re-opened five of the dam's gates to relieve some pressure, fearing their strategy would backfire by flooding their own stronghold of Falluja. The decline of water levels in the Euphrates also led to electric power shortages in towns south of Baghdad, which rely on steam-powered generators that depend entirely on water levels. A spokesman for the Ministry of Electricity said the power supply from Musayab power station had decreased to 90 megawatts from 170 megawatts. Government officials and advisers warned that ongoing closure of the dam could affect irrigation of farms in many southern provinces that depend on the Euphrates Rvier, including Hilla, Karbala, Najaf and Diwaniya [46].

In June 2014, fighters with the Islamic State in Iraq and Syria were advancing on the Euphrates River towards the Haditha Dam, the second largest in Iraq; located 120 miles (240 km) north west of Baghdad and the second-largest in Iraq. This action raised the possibility of catastrophic damage and flooding if they could take control of the dam. They already reached Burwana town, on the eastern side of Haditha town; six kilometers south of the dam site, and government forces were fighting to halt their advance. At one point, alarmed army officer told employees to stay inside and be prepared to open the dam's floodgates if ordered to flood the town and villages around. This would not be the first time that dams have figured in the conflict. In April, when ISIS fighters seized the Falluja Dam. They opened the gates, flooding crops all the way south to the city of Najaf. The water at one point washed east as well, almost reaching Abu Ghraib, close to Baghdad [47].

The security forces repelled ISIS fighters from Haditha later on in July [48].

In the same way ISIS had captured Ramadi Barrage north of the Iraqi city of Ramadi upstream from Falluja town and closed off its gates cutting water supplies to pro-government towns downstream and making it easier for its fighters to attack forces loyal to Baghdad. ISIS militants were opening only two or three of the dam's 26 gates for brief periods daily. This move was to prevent river water overflowing from ISIS' side of the dam, and also to allow some water to flow downstream toward ISIS-held Falluja. Water level in the Euphrates River was so low that the river could be walked across, making it easier for ISIS militants to cross and attack the pro-government town of Khalidiyah as well as the large security forces base at Habbaniyah. The level of the Euphrates River dropped by one meter near Amiriyat al-Falluja [49].

Similarly, the area around the Thartar Dam and the dam site itself near Falluja town, 98 km west of Baghdad, were the sites of many fierce battles between ISIS fighters and government forces. The main dam controls the flow from Lake Tharthar reservoir to the main feeder canal returning the flow to augment the Euphrates River flow in summer, and branching to do the same thing to the Tigris River flow. The militants could use the dam and the two other regulators on the feeder canal to inflict damage by flooding areas in Bagdad and Falluja. In one occasions, a U.S. official said intelligence reports suggest the extremists had opened at least one of the dam’s gates, although darkness has hampered efforts to determine how much flooding had resulted. All this led to many offensives to retake the dam and the area around it. In one of these operations, an army general was among the casualties, and 40 soldiers were taken captive by the militants [50].

One of the failed attempts of the terrorist group to capture a major dam and use it as a weapon of mass destruction was their attack in February 2015 on Adhaim Dam located 133 km northeast of Bagdad on Al-Adhaim River; a tributary of the Tigris River. At least 18 people, including 14 militants, were killed and 23 others injured as Iraqi security forces repelled ISIS. The attack occurred around Saturday midnight when dozens of ISIS militants, including suicide bombers, attacked the dam site but retreated after four hours of clashes [51].

Out of all these cases, the occupation of Mosul Dam site in August 2014 remains probably as the worst of all the other mentioned incidents.

Mosul Dam is located in north of Iraq on the Tigris River just 40 km north of the city of Mosul. It is the largest dam in Iraq, and it controls the Tigris River flow on its entry from Turkey with an active storage of 11.11 billion cubic meters of water.

The fall of Mosul city to the insurgents between 4 –10 June 2014 marked the beginning of a bloody saga in and around the city and opened the way to capture of Mosul dam on the 8th August 2014. The world was then holding its breath and waiting to what could have been the worst and most destructive man- made flood in history. This situation prompted; however, a swift and determined action to liberate the dam on the 18th of the same month from ISIS hands by the Iraqi forces and Kurdish Peshmerga supported by heavy air strikes carried out by the US air force [52] and [53].

The dam site was taken back, and it was saved from serious damage, but this case taught everybody a lesson to secure this type of structures at any cost. As a target with special important Mosul Dam was tagged by the international media as a weapon of mass destruction. It was fortunate that the terrorist group did not have enough time to plan and execute demolition acts, and the reservoir was at its lowest level at the time.

One more reason for abstaining from this could have been that ISIS did not wish to flood the city of Mosul which they had already occupied and which they considered as a great prize and the second capital of their Islamic State.

This situation led; however, to suspend the maintenance grouting operations that had continued for 30 years for more than one year. The continuous grouting operations were considered as the only possible mean to secure the dam against failure due to the continuous dissolution of gypsum in its foundation. In fact, an intensive study based on field measurements was performed during 2015 by the American Army Corps of Engineers who showed that the foundation was deteriorating at a very fast rate in view of halting of the grouting works; a case which led finally intensive efforts to continue the grouting operations that were resumed in the first quarter of 2016 [54].

5. Summary Points and Lessons Learned

Although dams’ safety is characteristic of design, construction and host of many other various elements of pure technical nature, dams’ safety is also influenced by “Human Factors” which can become important issues in this matter. Proper management based on good knowledge, and expertise can mitigate safety levels thus reflect positively on dam conditions and reduce a safety hazard to a minimum. However, variety of these human factors can also impair this safety, which reflects the negative aspects of the “Human Factor”. In this paper, we have tried to distinguish between two types of accidents involving dams which are related to human factors. The first are “Normal” accidents, which are related to unintentional mistakes and errors committed by site operators or remote controllers in operating one or more dams as a system. The second are what we have called the “Extraordinary” accidents; meaning they are caused by human actions for purposely destabilizing dams after thoughtful and carefully meditated decision making process. These are manifested in sabotage, terrorism attacks and acts of war.

Normal human errors and mistakes which lead to magnifying dam safety risks can be attributed to; personnel ignorance or negligence, or even still to overconfidence, insufficiently trained personnel or personnel that are unaware of risks and their lack of real time information ,which can exasperate the risks. The complexity of situations and lack of clear instructions may lead to risky situations also, and generally, human limitations due to fatigue, emotions, indifference and over stress can cause such dangerous conditions.

Lessons learned from such accidents due to human failings are, to have only highly trained personnel in charge of operation of dams, to simplify operation procedures as much as possible and provide clear instructions and operation manuals, in addition to following up all the instrumentation measurements and make sure of the good working conditions of the respective instruments by constant inspections.

In modern procedures, ITC applications have been used in operation of various systems, including dams. Decisions in many instances are made depending on the Supervision, Control and Data Acquisition (SCADA) systems without directly seeing the structure(s). Pitfalls in control software may appear in untried situations; similarly, an unexpected error in one element of the software can cause dangerous conditions. In such cases, alarm signals might be wrongly interpreted, or operation commands are not correctly received. Complexity of situations in site or remotely controlled system of dams can be exasperated by IT system failures. All such things can happen within the normal technological routines or to say normal human factors.

Lessons learned from dam incidents of such nature necessitate taking all precautions to have fool proof software with capacity for detecting and correcting errors, to have feedback capabilities and avoid consequences of IT system failure by having more than one channel for communication.

The use of modern technologies in controlling the direct operation of one dam or the remote control of a system of dams can lead in some cases into quite dangerous situations if these systems are not well guarded against external intrusion. This may result from cyber-attacks aimed at disrupting the normal operation of the system and leading to substantial deviation from the operational state as per design intents. Creating such dangerous situations can result from accidental or intentional misuse of the (ICT) technologies used for the dam(s) control, or can be a consequence of hackers' action infiltrating the system through loop holes which the owner was not been aware of.

Lessons learned are; to limit the number of the (ICS) users who have administrator access and; to reduce the extensive number of group accounts and limit that to the bare necessary; to make sure of the compliance with password policies by authorized users; and to remove inactive system administrators accounts. All these actions are to be done for stopping hackers who may have malicious intentions from creeping into the system. Moreover, it may be necessary that personnel with elevated system privileges should be subjected to complete and more rigorous background investigations. On top of this the mental health of authorizes users should be given enough scrutiny, and they should pass thorough psychological examination to bar entrance to the system to those which might have psychological disorders such as suicidal inclinations. This may be based on incidents, which have occurred in other fields activities and reminds us of Germanwings flight 9525 on 24th March 2015 when the 28-year co-pilot Andreas Lubitz had used the flight management system voluntarily to start the fatal rapid descent of the aircraft and the eventual crash into the French Alps killing himself and the150 passengers and the crew on board [55].

In more willful human actions aiming at dams' safety and causing "Extraordinary" incidents are acts of war, terrorist attacks; and sabotage work.

Destruction of dams during times of war has been one big source of concern to governments which stems from the many cases of dams' destruction in wars throughout history. Water resource systems including dams were targeted by combatants, including nations or states, and the intentions were to use water as a weapon to occupy an area or to bar the enemy from that area. Added to these in recent wars, the other objectives were inflicting maximum human losses, in spite of the 1949 UN Geneva convention, ratified in1977 on the protection of civilians in times of war [56]. Other intentions were paralyzing the enemy by disrupting his industrial production supporting the war effort as in World War II. This was also done in many late instances as in the 1991Gulf war and the occupation of Iraq war in 2003. These acts were done by the same countries who claim their advocation of human rights and upholding the International Law. But regrettably, these cases show clearly that human greed and want for vengeance have no limits.

Lessons learned as they became clear are, however; that using dams as tools in wars affects, mostly innocent civilians and cannot reverse the outcome of any war. Governments should abstain from such acts under any pretext or excuse, and all shall come to agree that these acts should be covered by the international humanitarian law, the law of The Hague, which addresses the various types of weapons and their permissible uses, as well as the behavior of combatants during hostilities; in addition to the Geneva rules, which, deal with the humanitarian treatment of the victims. Some of the underlying principles of the laws of war referred to are also the laws of armed conflict in a broader sense; that is the wanton destruction of human life, and property is prohibited.

Terrorist and saboteurs' actions, as we all know, have posed in late times another source of concern over dams' safety. The terrorist and saboteur's objectives have been to hold innocent civilian populations as hostages to force the government into submission to their political agendas using various means, including taking hold of dams. It is also clear from their actions and mentality that human madness sometimes goes further to even self-destruction.

In the present atmosphere of expanding use of the Media, terrorists and terrorist groups have also exploited the internet and social media not only to commit terrorist acts, but also to facilitate a wide range of terrorist activities, including incitement, radicalization, recruitment, training, planning, collection of information, communications, preparation, and financing [57].

As for putting an end to these follies, lessons learned are: Governments must work hard to eliminate the root causes which can emanate from social injustice or from fanatical religious ideologies; Governments shall, therefore, improve theirs record on human rights, improve living conditions and upgrade their education systems to match up with the challenges presented; Governments, moreover, are required to take extra care in the physical protection of dams and water resources infra structures and avoid any lax in their security. Finally, Governments should intensify and accelerate the exchange of operational information concerning the use of ICT systems by terrorist groups denying them any abuse of these systems.

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Dams Safety: Inspections, Safety Reviews, and Legislations

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Abstract

When a dam is built, its safety becomes a constant concern for the owner, the public and for governments. Therefore, continuous observation through routine inspections and safety reviews become necessary. Acting as protectors of public safety, governments and professional organizations save no effort in the promulgation of legislations and laying out guidelines for such inspections and reviews. These issues are discussed here starting with the basic first step of visual inspections by the operators and the follow up of detailed safety reviews by specialists. Careful visual inspections assisted by instrumentation measurements may reveal an early negative issue such as, but not limited to, increased seepage, increased uplift pressure, signs of weakness like cracking in the body of the dam, or dams' slope sloughing, and even damaged hydraulic control equipment. Documenting and reporting these observation helps in taking remedial measures in good time and may lead to more intensive safety reviews. Suggested check lists for the inspection engineers are given here, but these may be tailored for each dam according to its needs. These lists cover issues common to both embankment and concrete dams, and include other specific issues related to each type of them. Metal equipment take their share by listing such areas as corrosion, fatigue and cracking, tear, and wear and so on. Instrumentation measurements are also given their due consideration by giving brief mention of types of measurements needed and points

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to be observed in instrumentation control work. Finally, guidelines, rules, and legislations for Dam Safety Reviews are generally discussed giving examples from four countries in the world.

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1. General

Dams are built to serve useful purposes, but as all man-made structures they need constant attention to increase their service life, on one hand, and to remove any potential hazard they may present on the other. The assessment of dams' safety must be a continuing effort which requires the establishment of periodic inspection and examination routines followed by technical evaluation of any anomaly that may appear. Such evaluation becomes then the basis of any future action to eliminate the source of such anomaly and restore conditions to normal again. The extent and depth of this evaluation and remedial actions depend on many factors that contribute to the degree of safety of dams, these are:

- Dam's hazard classification with respect to loss of life and material losses.
- Size of the dam, its type, its height, and the volume of reservoir it impounds.
- Dam's behavior and performance during past years and the engineering judgment on its integrity.

The safety review of any dam takes special weight during its impoundment and just after first filling and shall continue in the following years after its commissioning. First filling of any dam reservoir may be considered as the most critical phase of the dam's life. The dam structures in this period adjust themselves to the newly applied loads. The foundation being subjected to new hydraulic loads and flows of water should adapt to the new conditions, while seepages through dam body should develop to its prescribed stable state and must be inside safe limits. At the same time, the body of the dam will apply its full weight on the foundation, uplift force being accounted for, the foundation must accommodate the new loads and settle within the calculated values expected, and as a whole the dam should adjust itself to any possible differential settlement. Not only that, but all appurtenant structures should operate in a smooth way as prescribed in their design.

Having passed the first filling phase, comprehensive safety reviews must continue periodically to ensure the application of the required safety standards, or in case of any deviation, to restore conditions back to normal. During the time between such reviews, continuous inspections and monitoring of the dam should be made at short intervals. All observations and obtained data including instrumentation recordings are to be tabulated and analyzed and kept for the next safety review; unless major threat is observed, in which case an urgent action is called for.

Strict inspection and review routines of dams' safety conditions were particularly re-enforced during the seventies of the last century by governments as a result of the major disasters at Malpasset (France), Vajont (Italy), and Baldwin Hills (United States); and in several countries new or revised laws for supervision of safety of dams and reservoirs were enacted.The Dam Inspection Act, U.S. Congressional Public Law 92-367, signed into law August 8, 1972, authorized the Secretary of the Army, acting through the Chief of Engineers, to undertake a national program of inspection of dams. Under this authority, the Corps of Engineers has (i) compiled an inventory of Federal and non-Federal dams; (ii) conducted a survey of each State and Federal agency's capabilities, practices, and regulations regarding the design, construction, operation, and maintenance of dams; (iii) developed guidelines for safety inspections and evaluations of dams; and (iv) formulated recommendations for a comprehensive national dam safety program.

The new French regulations require annual inspections of dams. Recognizing that the Malpasset and Vajont tragedies happened during reservoir filling, the French rules impose especially strict inspection requirements in the initial impoundment stage. They establish and regulate the rate of reservoir filling, and they call for weekly instrumentation readings, and require inspections at daily, weekly, and monthly intervals. Regular surveillance of the reservoir's peripheral areas is included. A report on the performance of the dam and reservoir is to be ready after 6 months.

In the United Kingdom, the Reservoirs Act of 1975 was written as an updating of 45-year-old rules that were put into effect after the British dam failures at Dolgarrog and Skelmorlie in 1925. The new law provides authority for regulators to intervene when an inspecting engineer's report has not been given adequate response. In such cases, the officials are empowered to effect the necessary corrective measures and to bill the costs to the owner. The British regulations also call for certificates which specify limits of safe reservoir operating level. An inspection is to be conducted by an independent qualified civil engineer not later than two years after issuance of the "final completion certificate," and at proper intervals thereafter. The Bureau of Reclamation considers two years to be the maximum time interval between field (onsite) examinations and six years to be the maximum time interval between evaluations. Also, Regional personnel make periodic examinations devoted primarily to operation and maintenance [1].

2. Scope of Dam's Safety Reviews

The frequency of these reviews depends on the age of the dam, last years' review reports findings and previously discovered problems, in addition to solutions taken to remedy them. As a normal interval, this may be once each five years for intensive reviews, but intermediate reviews may also be warranted in between. All this depends on the dam present conditions and its hazard classification.

Safety reviews have to be carried out by outside professional and specialized Engineer and includes disciplines of all branches of hydrology, geology and engineering relevant to dam safety issues.

The scope of any dam's safety review shall cover all issues, directly or indirectly, related to its location, its design, its construction, and performance and; so, may include, but not limited to the following work:

i. Design of the dam and its appurtenant structures is reviewed to assess their current performance compared to their intended performance and mark deviations, if any.

ii. Previous years' inspection reports are reviewed while original data and adequacy of the design criteria are checked to see if the behavior of the dam as indicated from these reports continues to be in conformity with required safety standards or not. Moreover, if these reports reveal changes of any of the loading conditions during the previous period, then behavior of the dam's structures is reviewed to be sure that such changes have not encroached on the factors of safety stipulated in the original design. In cases of deviation from the original factors of safety, new analyses have to be performed by incorporating the new set of data, and preferably using updated design theories and analysis tools and procedures to get the actual factor of safety. When serious dam safety concerns requiring immediate solution are discovered, then prompt action to bring up these concerns to the attention of the management must follow, and the solutions themselves have to be prescribed for mitigating safety conditions.

iii. Changes which occur in the environ of the reservoir may necessitate re-examination of the dam design or its operation and of the surveillance program. Hazards that worth studying are:

    1. developed surface of failure of reservoirs slopes resulting from the alternating filling and emptying procedure or after earthquakes,
    2. industrial activities carried out in the vicinity of the dam such as deep excavation, trenching, tunneling, building construction, or storage of explosives or flammable materials are also worthy of consideration.

As has been mentioned already, safety reviews shall depend, among other things, on the use of all the materials accumulated from onsite inspections; including visual observation of all structures made in conjunction with instrumentation monitoring to adequately assess the safety of a dam. Visual observation can readily detect indications of poor performance such as offsets, misalignment, bulges, depressions, seepage, leakage, and cracking. More importantly, visual observation can detect variations or spatial patterns of these features. Most visual observation provides qualitative rather than quantitative information, while instruments provide detailed quantitative information. Visual observation and instrumentation data are natural supplements and when taken together they provide the primary means for the reviewing engineers to evaluate the overall safety of a dam [2].

3. Areas for Dam Safety Considerations

Past accumulated experience and lessons learned from failures of hundreds of dams over past history highlight dam safety issues that should be examined and attended to in routine inspections and in safety reviews.

Generally, an important special area of concern is the geological conditions of foundation which may not be as competent as would be desired for the type or size of the dam. Deficiencies in the form of shearing planes, compressible or soluble materials, low peak or residual shear strength of seems of weak materials such as clay or faults filling materials my lead to various forms of potential failures mechanisms such as: large settlement of the dam and loosing freeboard, differential settlement and cracking, slides, increased seepage and excessive uplift. These consequences; however, may not be associated with the quality of the foundation rock only, but they can be exasperated by inadequate foundation treatment, or by regional subsidence caused by extraction of ground water or hydrocarbons, collapse of foundation soils caused by subsequent wetting. In many cases, seepage through foundation can cause piping of solid material or the dissolution and erosion of soluble material. An extensive treatment of dam safety issues related to geological factors has been presented recently in an article by the writer (authors?) which illuminates many aspects of such problems that can influence dams' safety [3]. Having an open eye for any anomaly stemming out of these factors during operation merits special consideration and study.

Another area of concern that needs the reviewing engineer's attention may have roots in hydrological and hydraulic factors related to the design and performance of spillways and outlet works. They can emerge from inadequacies caused by; underestimating the inflow design discharge due to either; insufficient hydrological data used in the design, or~~,~~ faulty hydraulic design. Overtopping in such cases is one possible outcome unless an auxiliary or emergency spillway is constructed. The emergency spillway itself should have adequate capacity, and in the absence of any proof of such adequacy, except for original hydraulic modelling warrants special attention by the reviewing engineer as such a spillway would be tried for the first time without previous knowledge of its efficiency.

Other problems can arise from restriction of tail water level by accumulation of gravel deposits and therefore reducing the outlet capacity or, in other cases the degradation of the downstream channel which may threaten and cause failure of downstream protection or wing walls. Stilling basins can accumulate rock, gravel, or debris and result in faulty hydraulic performance. In high velocity spillways or outlets, broken concrete or offsets in concrete surfaces can lead to dangerous cavitation and failure [4]. Pinpointing all such issues and suggesting their solutions remains as one of the main targets of any meaningful safety review.

Moreover, structural issues may lead to safety hazards due to cracking and displacement of concrete structures, while improperly designed or clogged filters and drainage layers, and obstructed drainage wells and underdrains can lead to excessive uplift and to sliding. Structural failure in a conduit, tunnel, or other conveyance structure causing obstruction of the flow or improper operation of the appurtenant structures can lead to overtopping. In many cases, rapid draw down of a reservoir may endanger the stability of the dam itself or the sides of the reservoir, and it may cause flooding of downstream communities. This can result from siltation of the reservoir and changes in downstream conditions. Safety reviews, therefore, have to detect possible changes in the reservoir and in the downstream conditions that may lead to changing the operation rules of the dam to eliminating sources of such dangers. Older dams should be given serious attention and need more frequent checks and reviews since their safety hazards tend to increase with time as consequence of materials deterioration, aging of equipment, and accumulated effects of aforementioned defects.

Last but not least, post-earthquake safety inspections of dams take special importance in establishing any changes of dam conditions and discovering damages; so to be able to draw up rehabilitation plans and remove causes of danger. General summary of dams safety concerns is presented in Table (1) for better clarification.

The outcome of inspections and safety reviews lead in many cases to mitigation actions; such as adding auxiliary spillway, dredging of the downstream reaches, or modifying outlets stilling basins, and can lead to extensive upgrading and repairs which prove the value of careful inspections and through safety reviews.

Table 1: General areas of Dam Safety Concerns

	Area of Examination	**Type of Dam**	**Source**	**Remarks**
1	**Design**	**All**	Incomplete or incorrect design data or criteria,	
			Incomplete foundation investigations.	
			Failure to define all probable loading conditions and failure modes	
			Incomplete materials investigation and testing.	
	Construction	**Concrete**	**Improper or incomplete foundation treatment, whether by using cutoffs or drainage wells; this will lead to excessive seepage and uplift on the dam base and may lead to erosion and suffusion of foundation or can result in sliding failure**	**This possibility is highlighted if foundation contains shearing planes or contain fines or soluble materials**
			Incorrect choice of aggregates leading to alkali reactivity and sulfate attacks. wrong methods of placemen and joints treatment	
		Earthfill	**Improper or incomplete foundation treatment, whether by using cutoffs or drainage blanket; this will**	**This possibility is highlighted if foundation contains**

			lead to excessive seepage and uplift on the dam base and may lead to erosion and suffusion of foundation	**shearing planes or contain fines or soluble materials**
			Failure to get the required compaction densities,	
			Failure to control placement water content, which can cause high settlement or differential settlement, piping, and shearing failures in embankment dams,	
2	**Reservoir Margin Defects**	**All**	**possibility of landslides formation due to faulting**	
			Seismic or landslides induced waves	
			Reservoir shores erosion	
			Reservoir's rim tightness and reservoir failure due to piping.	

4. Routine Inspection as First Step in Dam Safety Monitoring

4.1 General

Visual examination, measurements and routine checking of dams' performance are important parts of the dam monitoring work. Such monitoring is the first step in any safety evaluation process of the dam. This examination taken together with the follow up of instruments recordings and other measurements should bring to the attention of an experienced inspector any anomaly or unusual behavior of the dam and its appurtenant structures.

Such things which may be observed are unusual settlement, discoloration or increase of seepage water, any new wet spots or leaks and embankment sloughing. Such occurrences should bring alarm to the inspector who should not only describe and document such cases in detail, but act promptly to bring them to the attention of the higher technical level of management. In many known cases, observation programs were diligently performed but subsequent neglect and inaction resulted in further incidents and damage which could have been averted. The selection of inspectors is of paramount importance since experience and diligence of the inspector can play greatly in avoiding major incident in time.

Safety of dams can be threatened by so many factors, and the inspector should have good idea of possible failures and their consequences. This is important so that any observed anomaly can be tied down to its root cause, and that it may be subjected to further careful and intensive observation and follow up as would be needed. Generally, a list of the possible areas of concern may look like the one shown in Table (2). Such list is suggested only as a guide to possible events that can come about during the lifetime of the dam. It is the duty of the designer to include in his technical report of the dam all the assumptions used and the allowable limits of deviations compatible with safe performance; while the operation and maintenance report shall include best ways of carrying out visual inspections and monitoring of measurements, in addition to their frequency, details of the instruments used and the handling of the obtained data. The technical operating staff, in their turn, can upgrade this with the passage of time adding their experience with past occurrences and observed anomalies.

Table 2: Check list for types of failures and their causes; to be used by inspection engineers.

	Failure	**Cause**
1	**Foundation Deterioration**	**Removal of Solids and Soluble Rock Plucking**
		Under cutting
2	**Foundation instability**	**Liquefaction**
		Slides
		Subsidence
		Fault movement
3	**Defective spillway**	**Obstructions**
		Broken lining
		Evidence of overtaxing available capacity
		Faulty gates and hoists
4	**Defective outlets**	**Obstructions**
		Silt accumulations
		Faulty gates and hoists
		Gate position and location
5	**Concrete deterioration**	**Alkali- aggregate reaction and sulfate attack**
		Freezing and thawing
		Leaching of cement
6	**Concrete dam defects**	**High uplift**
		Unanticipated uplift distribution
		Differential settlement **Differential displacements and deflections**
		Overstressing
7	**Embankment dam defects**	**Liquefaction potential**
		Slope instability
		Excessive leakage
		Removal of solids and soluble materials
		Slope erosion
8	**Reservoir margin defects**	**Perviousness**
		Instability
		Inherent weakness of natural barrier

4.2 Inspection of the Dam Body and Foundation

In this inspection, it is always good practice that the inspection engineer gets familiarized with the design of the dams and the assumptions and data used by the designer before commencing any inspection round. It is advantageous also to see if the structural analysis of the dam is satisfactory or maybe, there is room for improvement if updated methods of analysis were to be used. A model check list of points to follow in such inspection is presented in Table (3).

Table 3: Model Check list for dam body and foundation Inspections.

	Location	Dam Type	Point for Observation	Remarks
1	Dam body and Foundation	Eartfill Rockfill Concrete Masonry RCC	Is leakage of water excessive? Is it increasing or decreasing? Is it clear or turbid? Are there large variations in individual drain discharges? Is potentially dangerous seepage apparent in the vicinity from sources other than the reservoir, such as in the abutments at high level?	Measurement of seepage water quantity are made by use of weirs or flumes. Quality of seepage water is checked by chemical analysis (See also Points; 2, 3, 5, 6, 7)
2	Dam body	Earthfill Rockfill	Is piping evident, especially where fills have been placed against or covered by structures?	Appearance of turbid water and/or boils are signs of internal erosion or suffusion
3	Foundation	Eartfill Rockfill Concrete Masonry RCC	Are uplift pressures within the design assumptions, or is it necessary to drill more relief holes into the foundation.	Such thing may appear from observation of piezometers
4	Concrete facing slabs	Earthfill Rockfill RCC	Is there visible warping or other distress?	This can indicate differential settlement in dam body
5	Dam body and Foundations	Earthfill Rockfill	Are there any signs of erosion of the embankment or its foundation?	Has cracking developed in structures, embankments, or foundations?
6	Dam body	Eartfill Rockfill Concrete Masonry RCC	Has cracking developed in structures, embankments, or foundations?	These have to be studies to see the originating cause
7	Foundations	Eartfill Rockfill Concrete Masonry RCC	Is there evidence of chemical alteration of foundation materials? Is there evidence of dissolution of foundation rock by seepage?	Testing of seepage water samples takes special importance for dams of karstic nature

4.3 Inspection of Appurtenant Structures Performance

In this part of inspection and checking procedures, the inspection engineer has to go back, as a first step, to the original or updated rating curves of reservoir, spillway(s) and other outlet works to satisfy himself on the conformity of these curves with the actual work of these structures to avoid any unpleasant surprise specially during floods. Visual inspections proving that the structures performance is not impeded by sediments and debris have to be carried out. But, one suggested list of points to be observed in a detailed inspection may look like, but not limited to, the list shown in table (4). Such list may be modified by an experienced dam safety reviewing engineer case by case depending on the actual situation.

Table 4: Model Checklist for Appurtenant Structures Performance.

No.	Structure	Dam Type	Point for Observation	Remarks
1	**Intake channels**	**All**	**Is the inlet channel free from obstructions and does not cause heading up of reservoir water level?**	**Heading up can encroach on the free board causing overtopping.**
2	**Spillway/ Outlet**	**All**	**Is the spillway/ outlet capacity adequate to lower the reservoir rapidly during an emergency?**	**Reference shall be made to the rating curves.**
3	**Spillway/ Outlets**	**All**	**Is the spillway intake capable of passing flood flows projected on the basis of up to date hydrological records?**	**Reference shall be made to the rating curves and updated hydrological study.**
4	**Spillway chute**	**All**	**Is the spillway chute in good working conditions and free from blemishes that can cause cavitation in conditions of fast flowing flows?**	**Visual inspection can prove this or otherwise.**
5	**Stilling basins and stilling pools**	**All**	**Is there danger of spillway discharge undercutting the stilling basin structure or stilling pool?**	**Visual inspection can prove this or otherwise.**
6	**Intake of outlets works and Spillway Intake**	**All**	**Are intake works for outlets and spillways free from silt and debris?**	**Visual inspection can prove this or otherwise.**
7	**Discharge outlet Channel**	**All**	**Is the downstream outlet channel constructed and maintained so that there will be no dangerous erosion, or debris deposited, in the river channel?**	**Decided from experience of channel performance and visual inspection.**

4.4 Special Points to be observed for Concrete Dams Inspections

In addition to points of interest for the inspecting engineer already mentioned in tables (2) and (3), there are other special points concerning concrete dams and should not escape either his attention or his careful documenting. In concrete dams, examination of concrete strength is an important parameter to be checked, especially in older dams. In freezing climate, additional thrust of ice sheets, over and above, the assumed thrust has to be reconsidered.

Carrying out nondestructive tests may reveal reduction of concrete strength, therefore, testing by extracting cores to confirm the integrity of the dam has to be conducted.

Among other checking, deterioration of concrete quality can result from the reaction of alkali- reactive concrete constituents with water over long time or from sulfate attack of the sulphates in cement and sand originally used for construction. Cracking or spalling will follow such reactions, but they may take some time to show their impacts. So, they are more characteristic of older dams. The final outcome is marked by reduction of concrete quality and strength which qualifies for effective upgrading measures to insure their safety. Alkali reaction and sulfate attack are among the most serious sources of undermining for concrete dams' safety to be attended to in rehabilitation programs of dams.

Other issues which can undermine dams' stability and integrity are opening up of joints between concrete lifts which means the performance of the dam as one unit no longer exists. The progress of joint opening shall be followed carefully and continuously and if seepage water starts to appear in these joints, then this indicates the development of dangerous uplift which represents another source of worry, and similarly may lead to leaching of cement and cause its crumbling. Freezing and thawing in extreme weather condition exasperate the process and lead to the progress of opening up of these joints.

Concrete surfaces subject to flow of water wear with time, especially in parts of high velocity flow in spillways and outlet structures. This wearing processes is exasperated by heavy sediment loads and debris carried by the flowing water during high floods.

The inspection engineer has to look further for indications of any cavitation problems which develop with time due roughened concrete surfaces or miss-alignments of planes in conduits, chutes, and stilling basins.

The question of workable drains and uplift pressure relief wells is important for the general stability of the dam. These devices which ensure dam's stability against sliding and overturning tend to be clogged during the passage of time by soil particles or salts depositions such as chlorides or sulphates.

Any meaningful rehabilitation program of concrete dams aiming at reducing its hazards has to be based on sound and careful review of inspections and testing results. Case histories of similar damages, inspection rounds and upgrading rehabilitation programs are given in a paper by the authors under the title of "Dams Safety and problems in Aging Dams" which is in the print [5].

4.5 Inspection of Systems and Metal Equipment

General but essential items involving systems and metal parts have to be checked on the onset of any safety inspection of dam before going into the next detailed procedures. These general items are summarized in Table (5).

Table 5: General Issues of Safety Requirements for installed systems

No.	Item	Points for Observation	Remarks
1	Auxiliary Power Supply	Is it available and in standby operating readiness?	This covers generators and connecting cables, switches, and controls.
2	Systems availability for manual operation of hoists and servomotors	Oil filled cylinders and their state of readiness.	These equipment are essential for controlling out flow and need constant attention.
3	Adequacy and reliability of ventilation system provided in shafts, tunnels, and galleries	Working readiness and reliability.	To prevent corrosion and to protect personnel from noxious gases.
4	Gates, Valves , Hoists and Servomotors and other essential operation equipment	Are they in operable conditions?	Frequent checks are necessary.
5	Drainage sump pumps	Are they in operable conditions?	Frequent checks are necessary
6	Automatic alarms and Telemetering devices	Are all properly functioning?	Constant Checking is needed to avoid false alarms or readings
7	Operating mechanisms which do not need frequent checking	Their functioning still need to be checked every now and then. Such examples are firefighting systems and the like.	To ensure working condition when needed.
8	Emergency supplies and spare parts and tools	An inventory of these items shall be kept ready for use and shall be updated from time to times.	Such supplies and tools help taking fast actions during emergencies.

The detailed inspection of the spillway and outlets metal works and piping systems is an important aspect for insuring safe operation, therefore, they deserve special attention in the safety inspection routines. Such inspection should be followed by corrective actions in case of discovering deficiencies.

Common areas of problems which should not escape the attention of the experienced inspecting engineers may be outlines in the following:

Corrosion and Rusting, and Use of Galvanic Action

Corrosion is progressive deterioration resulting from exposure to moisture, acid, and other corrosive agents, and usually is marked by scaling or flaking, pitting, and color changes. Loss of paint or other protective coatings can leave a metal surface subject to corrosion, especially if the surface is cycled between wet and dry condition. Unchecked corrosion eventually leads to failure of a metal structure.

In the protection of metals such as those used in control equipment of gated spillways and outlets structures or protective trash screens and piping systems, cathodic protection may prove to be necessary. By installation of these systems, a galvanic anode, a piece of a more electrochemically "active" metal is attached to the vulnerable metal surface exposed to an electrolyte. Galvanic anodes are selected because they have a more "active" voltage than the metal of the target structure which are typically steel. Normally, anodes consist of such metal pieces or bands like Zinc or Magnesium attached to the metallic part needing protection. The inspector has to make sure of locating and examining all accessible anodes installed for cathodic protection of the metal parts under consideration. An anode may need replacement if it is excessively corroded.

Fatigue and Cracking

Fatigue is loss of metal strength from repetitive loading, such as being bent back and forth. Protrusions on metal components or components with moving parts are most likely to suffer fatigue. Cracking is usually induced by vibrations and fluctuating dynamic loadings common in hydraulic structures. Distortions or cracked paint or damaged welding seem may indicate locations where a metal structure suffers from fatigue. The deterioration continues until the affected area cracks and/or breaks. Visual observation of such distortions and loss of paint or distorted welding seem should not escape the attention of a good inspector nor the appreciation of its consequences. One step further to such inspection and discovery of such defect may be taken by applying appropriate nondestructive tests in order to reach decision on the best possible repair action or may be the replacement of the defective part.

Erosion, Tearing and Rupture

Flow surfaces and areas around rivets and splice plates may be scoured by abrasive debris. These items may not be too difficult to replace when are discovered by the observant eye of practicing inspector. Discovering such issues and treating them in time could save much trouble later if they escaped notice or being ignored. Tearing and rupture may result from impact of large debris such as a logs slamming into a steel lining. In spillways, metal components are most likely to tear and rupture during storms or other occasions when the flow is heavily loaded with sediments and debris. Tears and ruptures can cause a metal structure to fail completely, or expose the structure to corrosion, cavitation, fatigue, or other damage(s).

Cavitation

Cavitation of metal surfaces, such as metal conduits, can occur when high flow velocities exist on flow surface with offsets and irregularities. The bubble collapse dynamics of cavitation cause pitting of the surface, which leads into progressive deterioration to the damaged area. As in concrete, the site of initial cavitation damage triggers the formation of another site downstream, so the process develops in a leapfrog pattern. Areas just downstream of gates and valves are susceptible to this phenomenon also [6] and [7].Timely discovery of cavitation by careful inspection can save a great deal of cost and repairs effort in selecting and applying the corrective measure(s).

4.6 Program of Inspections and Reports of Findings

To maintain control of the inspection program, an inspection schedule should be maintained which lists each feature to be inspected, frequency of inspections, date of last inspection, maintenance record, description of repairs made, and date of next inspection. The schedule should also have a note on major alterations that are made. Inspection personnel should be selected carefully; should have qualifications commensurate with their assigned levels of responsibility and have received training in inspection procedures. Qualifications and training required for inspection personnel may vary with the complexity of the facility and with the level of inspection.

Upon completion of an inspection round of a dam, the inspector' team should formulate and report its findings in the Inspection Report.

When serious dam safety issue is encountered then this must be brought up immediately to the attention of the dam management for further action.

This report serves also as a follow up of previous inspections and should have answers to whether all previous adverse or questionable conditions have been taken care of, and if deficiencies been remedied without delay. The question whether the operation personnel have proper instructions and authority for actions to be taken during an emergency shall be addressed.

A good report has to contain a follow up of developments such as variations in seepage water quantities and piezometric measurements related to changes in the reservoir water levels. The report shall, therefore, contain summary of such measurements and recordings taken during the time period since the last inspection added to those produced in the previous report. Presentation of such measurements shall be in tabular and/or graphical form with appropriate comments on them.

If an emergency had occurred during the period between inspections, whether in passing exceptional flood or an earthquake event, detailed reports on these incidents shall be attached as appendices, even if such reports were prepared by other specialized teams or agencies who, without any doubt, have included observations, descriptions and evaluations of damages that were sustained by the dam and its structures, if any.

All noteworthy observations should be documented by photographs, videos, and illustrations with sufficient details of dates and comments. Finally, the report shall evaluate the adequacy, or otherwise, of periods between inspections and if more frequent inspections are needed in light of dam safety status.

4.7 General Types and Frequencies of Inspections and safety Reviews

Types and frequency of dam safety inspections vary from country to country, but generally they follow general agreed principles which take into consideration type of dam, its age, and the risks it presents to population and infrastructures.

Inspection types and their frequencies included in an inspection program for any dam should not preclude other inspections or more frequent inspections, if deemed necessary depending on dam history and importance of the facility. For some projects, less frequent inspections may be permissible where hazard potential and structural integrity warrant such relaxation.

A model procedure for carrying out inspections and their types are presented in the following:

Informal inspections and their frequency

Informal inspections are normally assigned to onsite operating personnel who have direct responsibility of the smooth operation of the dam and follow up of its good conditions. Those operators need not have formal education in the fields of engineering or geology, but they should be well trained and made aware of the heavy reliance placed upon them and the great importance and absolute necessity of their careful inspection and reporting.

They are to identify and report the various aspects of dam behavior in accordance with detailed instructions and checklist of items needing inspection.

This list should be prepared specifically for the project by engineering and operating specialists and should give adequate instructions and guidance.

Any unusual conditions that seem critical or dangerous and needs immediate attention should be reported promptly to the higher level of management or to those assigned the inspection responsibility. Particular attention should be given to detecting evidence of (or changes in) leakage, erosion, sinkholes, boils, seepage, slope instability, undue settlement, displacement, tilting, cracking, deterioration, and improper functioning of drains and relief wells, as outlined in previous paragraphs.

Informal inspections should be scheduled as needed according to the dam's size, importance, and potential for loss of life and damage to property.The schedule for inspection should be changed by the engineers as required to be responsive to observed changing conditions. But the operating personnel are expected to perform an inspection immediately after any unusual event such as large floods, earthquakes, suspected sabotage, or vandalism. In such cases they may need to take photographs and/or videos to give real time recordings of such incidents.

Intermediate Inspections and their frequency

Intermediate inspections should include a thorough field inspection of the dam and appurtenant structures, and a review of the records of inspections made at and following the last formal inspection. If unusual conditions are observed that are outside the expertise of those inspectors, arrangements should be made for inspections to be conducted by specialists.

Intermediate inspections frequency is normally specified in various country dam safety codes and guidelines, but preferably performed on an annual basis, or at least biannually, especially where there is a high probability that dam failure could result in loss of life. For other dams, intermediate inspections should be scheduled by responsible engineers on the basis of the dam's size, importance, and potential for damage to life and property.

Intermediate inspections should be performed by technically qualified engineers, experienced in the operation and maintenance of dams, and trained to recognize abnormal conditions. The inspectors should have access to and, be familiar with, all permanent documentation, especially the operation and maintenance histories for the dam and should be responsible for, and intimately familiar with, the operating characteristics of the dam. The dam operators should be participants in these inspections.

Formal and Special Inspections

A formal inspection is required periodically to verify the safety and integrity of the dam and appurtenant structures. Formal inspections should include a safety review to determine if the structures meet current accepted design criteria and practices. It shall also involve review of all related documents including instrumentation records, operation, and maintenance procedures and, to the degree necessary, documentation on investigation, design, and construction.

In performing the detailed inspection of the dam appurtenant structures and equipment, diving inspections of underwater structures affecting the integrity of the dam may have to be included. All formal inspections should be conducted by a team of highly specialized and trained engineers and geologists. To assure that a dam and its appurtenant facilities are thoroughly inspected, checklists should be prepared to cover the condition of foundations, structural, electrical, and mechanical features.

Such inspection should also verify that operating instructions are available and fully understood. To answer whether instrumentation system is adequate and data is assessed to assure structures are performing as designed, and if there are emergency provisions for accessing to and communication with all project operating facilities.

Formal inspections should be made periodically at intervals not to exceeding five years. Depending on past experience or the project history, some dams may require more frequent formal inspections. Out of the adopted inspection schedule, special inspections should be performed immediately after the dam has passed unusually large floods and after the occurrence of significant earthquakes, sabotage, or other unusual events reported by the operating personnel.

Formal and special inspections are being conducted under the direction of licensed professional engineers experienced in the investigation, design, construction, and operation of dams. The inspection team should be chosen on a site-specific basis considering the nature and type of the dam. The inspection team should comprise individuals having appropriate specialized knowledge in; structural, mechanical, electrical, hydraulic, and embankment design, geology, concrete materials, and construction procedures. They should be capable of interpreting structural performance and relating conditions found to current criteria and safety aspects. It is imperative that the inspection team adequately prepare for the inspections by reviewing and discussing all documents relating to the safety of the dam [8].

5. Instrumentation Monitoring and Surveillance

As supplement to field inspections, instruments' observations and recordings can give better insight to dam's behavior; together with the inspections they also provide sound basis for dam safety reviews. Instrumentation to monitor structural and functional performance are therefore necessary.

Careful examination of instrumentation data on a continuing basis may reveal a possible critical condition. Conversely, instrumentation may be means of assuring that an observed condition is not serious and does not require immediate remedial measures.

Design and installation of instruments are usually done during design and implementation phases. The number of instruments, their types and their layout are decided by many factors, among these; the importance and size of the dam, the magnitude of risks it poses to population and infrastructures in the downstream reach in addition to complexity of the site.

The types of instruments needed in typical important dam, depending on type of dam whether concrete or embankment dam, may include combination of any of the following devices:

- Piezometers
- upstream and downstream water level gauges,
- horizontal and vertical alignment instrumentation (Concrete Structures), geodetic survey beacons,
- inclinometers,
- inverted plumb bobs,
- distofores and extensometers
- total earth pressure cells and pore-water pressure cells (Embankment Structures),
- settlement plates,
- seismic instrumentation,
- Seepage water quantities measuring devices such as Weirs and Parshall flumes are needed and frequent sampling and testing for soluble salts concentrations may be required in many special cases where dams are located in karst areas or built on soluble foundation rock.

Instruments should be examined periodically for proper functioning, and the adequacy of the installed instruments are to be assessed from time to time by specialists to determine if they are sufficient for evaluating the performance of the dam. An experienced inspector may find gaps in important data which have to be filled by more of the same instrument or by another type of instrument. If for example seismic instrumentations are not installed in a dam, the need for such instruments becomes evident after an earthquake event.

Inspection of the installed instruments' status and their working condition is the duty of the dam operation personnel, while final evaluation of the devices' conditions, quality of data obtained and need for more of them is the duty of an instrumentation expert working with the formal inspection team. Questions such as, but not limited to those, are given in table (6) have to be answered.

Table 6: Points to be observed in instrumentation control work

	Item	Point of observation	Remarks
1	**U/s and D/s water levels gauges**	**General working conditions**	**Are there any obstructions such as weed , debris or others**
2	**Stand pipe piezometers**	**General appearance and checking for any erratic reading**	**Can be damaged by external agents, or have clogged filters. In piezometers acting under hydrostatic conditions, pressure manometers have to be checked, recalibrated, or replaced**
3	**Relief wells**	**General appearance and quality of drainage water**	**Is drainage water turbid or discolored? Do chemical testing results indicate increase in soluble salts concentrations?**
4	**Settlement and alignments, horizontal and vertical displacement devices**	**Is there any out of normal movement in dam body and appurtenant**	**If measured values are out of calculated safe values, do these merit special in-depth analysis?**
5	**Performance and presentation of measurements and recordings**	**Are competent, trained personnel assigned to surveillance?**	**Is performance of this work done according to approved check list? Does the quality of presentation allows good interpretation**
6	**Timing of observations and reporting**	**Does the frequency of readings conform with safety appraisal requirements? Is reporting done in time?**	**Observation frequency is established by a specialists and may deviate in case of unforeseen emergencies as decided by well-trained operators. Reporting in time facilitates early safety review**
7	**Adequacy of installed instrument devices.**	**Numbers and types Quantity and quality of obtained data.**	**Is the retrieved data enough to ascertain the safe condition of the dam?**

The instrumentation data should be collected by personnel trained specifically for this purpose. Such training should include training to recognize and report immediately anomalies in the readings or measurements. Performance observation data should be properly tabulated for easy interpretation and may be supported by comments, diagrams, and curves.

The frequency of instruments' readings should be established at the time of design of the dam; when the instrumentation system itself is designed. Such frequency is established to give a timely warning of possible adverse conditions. But, whenever necessary, more frequent readings, sometimes as often as hourly, should be done to monitor a suspected rapidly changing adverse condition. The frequency or number of readings may be reduced after the project has been in operation for an extended time and performance observation data indicates that readings have stabilized.

It is essential that instrumentation data be processed, reviewed, and assessed in a timely manner by specialists familiar with the design, construction, and operation of the project. Operation manuals and design information should be referred to in the evaluation of possible adverse trends. The performance observation data should be periodically analyzed to determine whether project structures are reacting as assumed in the design, and to detect deviation if any.

Many dam regulating authorities; such as governments or other public authorities have laid out guidelines and procedures for carrying out inspections and safety reviews of dams under their jurisdiction to ensure their proper performance and reducing risks to the public.

Examples are given in references such as those listed in the references list under numbers [4], [8], [9], [10], [11] and [12].

6. Guildlines, Rules and Legislations for Dam Safety Reviews

Generally, rules for inspection, safety review and legislations in various countries may have taken different routes of development in different countries, but such development has been guided and dictated more often from experiencing failures and incidents in these countries together with lessons derived from other countries experiences. Most countries owning dams have promulgated laws and acts regulating these issues, such as UK, USA, France, Canada, India, Australia, Sweden, and others. In the following paragraphs, only the first four mentioned countries are considered which exemplify similar methods and procedures in all other countries.

6.1 Dam Safety Guidelines and Legislations in UK

Dam safety Management in the UK is governed generally by legislations whereby responsibility is divided between dam owners, councils or local governments, government departments and technical panels of qualified engineers appointed by the Institute of Civil Engineers (ICE). Promulgation of legislations leading to prescribing procedures for dam safety control was done for the first time in 1925. It was driven by the failure of two dams in this year which caused a flood that swamped the village of Dolgarrog in North Wales, killing 16 people.

The disaster began by the failure of the Eigiau Dam, a small gravity dam. The water released from the reservoir flooded downstream, and overtopped the Coedty Dam, an embankment dam. This dam failed, releasing the huge volume of water that flooded Dolgarrog. The disaster at Dolgarrog led the British parliament to pass the Reservoirs- Safety Provisions Act in 1930 that introduced laws on the safety of reservoirs. This has since been updated; the current legislation is:

i) in England and Wales: Reservoirs Act, 1975
ii) in Scotland: Reservoirs-Scotland Act, 2011
iii) in Northern Ireland: Reservoirs-Northern Ireland Act, 2015

These Acts cover the safety reviews of all reservoirs in the United Kingdom that can hold at least 10,000 or 25,000 m^3 of water, depending on location (currently the 10,000 m^3 lower limit only applies in Wales).

Under these Acts, there are "Panels" (or groups) of civil engineers who are responsible for checking the safety of these reservoirs and their dams. They are called “Panel Engineers”, and they can only belong to the Panels if they are considered to be professionally qualified, and experienced in reservoir safety matters. Every reservoir has to be safety reviewed every 10 years, or more often, when it is necessary. Panel Engineers also get involved with the construction of new reservoirs and repairs and changes to existing ones [13].

Annual safety inspections are left to a supervising engineer who is required with his team to supervise the operation and maintenance of the reservoir and produce an annual statement summarizing the performance of the reservoir over the previous year. A Supervising Engineer's team must be available "at all times". They can require maintenance works to be carried out and can also recommend that specialist engineers carry out an early inspection if they have sufficient concerns.

Although the "Supervising Engineer" is responsible for advising the owner for any issues that might affect safety of the reservoir, they will only usually visit once or twice a year, and therefore rely on technicians and operational staff to spot early signs of a problem. These are usually recognized by routine daily or weekly inspections as deemed necessary through visible changes to the structure, or by recording and taking measurements. Having been previously well trained and informed of what can go wrong, the operating team has an essential role in helping to look out and discover any early signs of problems with particularly spotting signs of change. Such possible changes include but not limited to, movements, seepages, and other features putting the dam at risk, for example a blocked overflow [14].

6.2 Dam Safety Guidelines and Legistlations in the United States

Dam safety legislations and inspection procedures in the United States have, so far, followed gradual development and were driven by series of catastrophic failures. History of dams in the United States is rich with failure cases which had caused tragic deaths and appreciable damages. Early examples are:

- Mill River Dam (1847), Massachusetts,140 people were killed,
- South Fork Dam (1889), Johnstown, Pennsylvania, 2,200 people were killed,
- St. Francis Dam (1928), California, 450 people were killed.

Many more failures in Arizona, Tennessee, Oregon, North Carolina, Texas, Virginia, West Virginia, and elsewhere across the U.S. occurred around the turn of the century, leading to the passage of some early state dam safety legislation.

The failure of St. Francis Dam, in March 1928, was a landmark event in the history of state's dam safety legislation, spurring legislation not only in California, but in neighboring states as well. However, most states had no substantive dam safety laws prior to a series of dam failures and incidents that occurred in the 1970s.

The conclusion that may be drawn from all these cases is that safety inspection procedures were not as rigorous as they should have been and in most cases this was left to the owners themselves, and there were no unified legislations governing this matter.

Major disasters of dam failures occurred in various countries during the 1960s such as at Malpasset- France on December 2, 1959, killing 423 people, Vajont- Italy on October 9, 1963, killing 2,000 people, and Baldwin Hills- USA on December 14, 1963, resulting in five deaths and the destruction of 277 homes, but without the vigorous rescue efforts much greater loss of life could have happened. These catastrophes instigated several countries to enact new or revise laws for supervision of safety of dams and reservoirs. The Dam Inspection Act, U.S. Congressional Public Law 92-367 signed into law on August 8, 1972, authorized the Secretary of the Army, acting through the Chief of Engineers, to undertake a national program of inspection of dams. Under this authority, the Corps of Engineers has:

(i) compiled an inventory of Federal and non-Federal dams;
(ii) conducted a survey of each State and Federal agency's capabilities, practices, and regulations regarding the design, construction, operation, and maintenance of dams;
(iii) developed guidelines for safety inspections and evaluations of dams; and
(iv) formulated recommendations for a comprehensive national dam safety program [1].

Following the failure of Kelly Barnes Dam, Toccoa Falls, Georgia, on November 6, 1977 killing 39 students and college staff and causing about $2.5 million in damages, President Jimmy Carter directed the US Army Corps of Engineers to inspect the nation's non-federal high-hazard dams.This "Phase (I) Inspection Program" lasted from 1978-1981. The findings of the inspection program were responsible for the establishment of dam safety programs in most states, and, ultimately, the creation of the National Dam Safety Program, which supports dam safety programs in 49 states, all but Alabama, which has yet to pass dam safety legislation [15].

Resulting from the Program, inspections of dams and number of Emergency Action Plans (EAPs) for high-hazard potential dams has increased, and better tools and guidance for dam operation and maintenance have been developed. The work that has been done under this Program has played an important role in reducing the loss of life and damage to property and the environment from dam failures.

Further to this, the Dam Safety Act of 2006 - (33 U.S.C. § 467 note)- required the (Administrator) of the Federal Emergency Management Agency (FEMA) to submit a biennial report on the National Dam Safety Program to Congress. The act required the report to include the status of the Program and the progress achieved by Federal agencies and participating states in implementing of the "Federal Guidelines for Dam Safety (FEMA, 2004)" [16].

In all this activity the USACE had carried out major dam inspections of approximately 700 dams nationwide that fall under its jurisdiction with the objectives of:

(i) Ensure the dam system will perform as expected,
(ii) Identify deficiencies or areas that need monitoring or immediate repair,

(iii) Continuously assess the integrity of the dam in order to identify any changes over time,
(iv) Collect information in order to make informed decisions about future actions,
(v) Determine if the dam is being properly operated and maintained.

Achieving these goals requires the United Army Corp of Engineers to conduct two types of dam inspections. The first one is the "Annual Inspection", which is performed on an annual basis to ensure the dam is being properly operated and maintained. The second one is "Periodic Inspection" which is the next level of inspection and is conducted by a multidisciplinary team led by a professional engineer. It includes a more detailed, comprehensive evaluation of the condition of the dam and will be conducted every five years. Components of the "Periodic Inspection" include evaluating annual inspection items; verifying proper operation and maintenance; evaluating operational adequacy, structural stability, and safety of the system; and comparing current design and construction criteria with those in place when the dam was built [17].

The United State Bureau of Reclamation in its turn has followed the "Safety of Dams Act" of 1978- Public Law 95-578, November 2, 1978- which provided the Bureau through the Secretary of the Interior with the authority to construct, restore, operate, and maintain new or modified features at existing Federal Reclamation dams for safety of dams purposes.
The Bureau of Reclamation is responsible for administering its 491 dams and takes care of wide range of activities which includes those related to the operation, maintenance, and safety reviews of these dams. The activities are:

(i) Periodic onsite examinations;
(ii) Following up dam behavior through observations and reporting;
(iii) In depth studies of visually identified or suspected defects such as general deterioration, seepage, structural distress, spillway and outlet hydraulic behavior, and adjacent endangering geologic conditions;
(iv) Comprehensive analytical studies to evaluate such items as spillway capacity, seismic stability, or surveillance instrumentation utilizing modern technology;
(v) Maintaining of up to date "Standing Operating Procedures" for operation and maintenance of each dam and appurtenant structures;
(vi) Awareness and reporting of hazardous conditions existing in upstream dams belonging to others that might adversely affect the safety of Bureau dams;
(vii) Monitoring of potential landslide areas;
(viii) Preparation of designs and supervision of construction for dam modification, and
(ix) rehabilitation, or replacement for safety purposes [4].

6.3 Dam Safety Guidelines and Legistlations in France

In France, the general rule is; protection of persons and property is a responsibility of the Government, who must legislate (create regulation) and enforce the rules through administrative bodies (agency, department, office,.. etc.) to provide for the safety and security of the people, property, and environment. That is why dam design, construction, rehabilitation, enlargement, alteration, operation, inspections, monitoring, maintenance, repair, breach, abandonment, and removal must rely on a legal frame that establishes rights, responsibilities and duties of the parties involved. Within this legal frame, three “ministries” have responsibility for dams. The Ministry of Transport; for structures associated with canals, the Ministry of Industry; for hydro and tailings dams, and the Ministry of Environment; for other dams.

The present regulation requires that safety concerns be addressed as early as the preconstruction stage. It is meant, as stipulated by the legislature, to cover dams and levees in a coherent manner, and the basic framework should be the same for all dams, regardless of whether they are for electricity production, water supply, irrigation, flood control or other purposes.

Requirements for dams’ Inspections, safety reviews and control vary on the basis of dam and reservoir size. Size is based on criteria combining dam height (H) and reservoir capacity (V), being intuitively true that a small dam impounding a lot of water may be as much of a danger as a high dam impounding little water.

Recent changes in French regulation made in 2007 require adoption of new dam classification system, according to “Risk Prioritization Criteria” introducing a graded system with four classes of dams:

A: $H \geq 20$ m
B: $10 < H < 20$ m and $H^2 \sqrt{V} \geq 200$ (V in ha^3)
C: $5 < H < 10$ m and $H^2 \sqrt{V} \geq 20$
D: $2 < H < 5$ m,
Reference [18]

Moreover, for each new dam with a height more than 20 m and/ or the size of the reservoir is 15×10^6 m^3, a technical file must be submitted by the owner to government authorities for approval and it should be subject to emergency planning regulations [19]

Inspections, safety reviews and repairs of old dams are normally overseen by a technical committee called the “Standing Committee”, which is formed of members from the previously mentioned ministries and acts as the “Supervising Authority”. The “Supervising Authority” is responsible for enforcing the “Regulation” and ensuring that the “Owner” complies with his responsibilities, this means that the staff in charge of the supervision must possess sufficient skills and knowledge to be able to judge if the “Owner’s” efforts are commensurate to his duties.

Dams under construction are also subject to compulsory continuous safety surveys by the construction supervisor, who is also responsible for the safe first filling.

After end of construction, the "Owner" must do periodical visual inspections and monitoring at agreed frequencies and should report the findings of any anomaly or defect. All these are considered as the first line of defense. Comprehensive safety reviews should be carried out by the "Owner" yearly during the first five years from the date of commissioning followed by similar checks at five years and ten years' intervals. The dam owner is also responsible for preparing an Emergency Action Plan (EAP) to meet unforeseen emergencies but inundation maps in case of dam failure are prepared by the local authority of the county in which the dam is located.

Initially the "Supervising Authority" and the "Owner" should conduct an inventory of dams to determine:

a. name and address of the owner;
b. the location, type, size, purpose, and height of the dam;
c. storage capacity;
d. as accurately as may be readily obtained, the area of the drainage basin, rainfall and stream flow records, flood-flow records, and estimates;
e. classification of the dam according to the hazard potential risk.

Based on this inventory, the "Supervising Authority" shall first classify the dams according to their risk (potential and present) and even identify the small dams with a high risk accordingly to the Risk Prioritization Criteria into classes of A, B, C, or D; as figured already, and then performs safety reviews of these dams. The regional supervisors shall have the support of teams of specialists (engineers with experience in civil, geotechnical, hydraulic, hydrological and risk engineering), and whenever, the "Supervising Authority" finds that any owner has violated a provision of the "Regulation" it may do the following:

i) issue an administrative order requiring any such owner or company to comply with his/hers/its duties,
ii) bring a civil action against the violator,
iii) levy a civil administrative penalty (fine),
iv) petition the Attorney General (or its equivalent) to bring a criminal action.

The use or application of any of these remedies shall not preclude recourse to any of the other remedies prescribed.

The "Supervising Authority" also shall have the power to adopt rules, standards and requirements for the design, construction, reconstruction, enlargement, alteration, operation, monitoring, maintenance, modification, repair, breach, abandonment and removal of dams and reservoirs to carry out the purpose of the Regulation. To exemplify some of the responsibilities of the "Supervisory Authority" in practical terms, Table (7) presents the duties of this body in France regarding the four classes of dams [18]. Further to this, the duties of dam owners as defined by the present French legislation are based on the same dams' types classification and summarized in Table (8) [18].

Table 7: Duties of the "Supervisory Authority" according into the French dam type's classification.

Duties	A	B	C	D
Technical and administrative approval	yes	yes	yes	yes
Attendance at foundation excavation acceptance inspection	recommended	recommended	possible	no
Inspection of completed scheme and verification of conformity	yes	yes	yes	no
Approval of operating surveillance procedures	yes	yes	yes	no
Periodic inspection	1 year	1 to 5 years	1 to 10 years	no

Table 8: Duties of dam Owners and the Concession Holders in France.

Duties	A	B	C	D
Complete record of dam design, construction, and service life	yes	yes	yes	yes
Register	yes	yes	yes	yes
Detailed technical inspection	1 year	2 years	5 years	10 years
Operator's report	1 year	≤5 years	5 ≤ years	no
Instrumentation report	2 years	≤5 years	5 ≤ years	no
Operating rules (normal operation and flood period)	yes	yes	yes	yes
New design or modifications to be submitted to Permanent Committee on Dams and Water Retaining Structures	yes	no	no	no
Ten-year safety review including full technical inspection	yes	no	no	no
Hazard Study	yes	yes	no	no
Report any significant events	yes	yes	yes	yes

As a result of the new regulations governing dam safety, priorities for repairs and rehabilitation works are based now on the inspection and safety reviews' programs prescribed already, and in view of limited budgets and resources, dam owners who operate and maintain large number of dams such as Electricite de France (EDF) has established the "Super Hydro Program", in which it planed more than 300 maintenance operations between 2007 and 2011, for a total cost of more than 500 million euros (US$716 million). Most of these operations involve gate or penstock work, such as replacing or refurbishing gates and replacing or painting penstocks in addition to several civil engineering works.

EDF operates about 450 hydro plants with construction dates stretch from the end of the 19th century to 2009, and the average age is about 60 years. In 2005 and 2006, several incidents at these plants had happened and results of inspections and reviews showed the need to better detect the progression of aging and to schedule suitable maintenance operations.

As a result of new 2007 regulation, EDF faces the challenge of producing 239 risk assessment studies and 149 periodic safety audits. These studies were due before the end of 2012 for dams classified as A and before the end of 2014 for dams classified as B [20].

6.4 Dam Safety Guidelines and Legislations in Canada

Guidelines and Legislations related to dams' safety in Canada, including the regulation on construction, operation, maintenance, and safety issues are provincial/ territorial responsibility unlike some other countries like, UK, France, Mexico, South Africa, etc…Canada does not have a federal regulatory agency or overall program to guide the development of requirements for the safe management of dams. The Federal Government, however, has regulatory requirement over some aspects such as approval of dams to be constructed in navigable waters and dams located on boundary waters with the US and dams constructed and operated by the Canadian's nuclear industry. The Federal Government also has regulatory interests through the Fisheries Act, Species at Risk Act, the Environmental Protection Act, and the Nuclear Safety and Control Act. Moreover, the federal government manages its own dams, which are exempt from provincial regulation such as Parks Canada Dams.

Provincial and territorial jurisdictions generally have their own regulatory framework and guidelines to inform the dam owners/ operators about roles and responsibilities, regulatory requirements, processes, and procedures; as well as methodologies for compliance assurance. The provinces of Alberta, British Columbia, Ontario, and Québec have varying degrees of specific regulatory requirements. Some jurisdictions may rely on legislation related to management of water resources and refer to industry best practices as published by the Canadian Dam Association (CDA). This association, a non-profit organization formed in the 1980s, provides dam owners, operators, consultants, suppliers, and government agencies with guidance on issues of dam safety in Canada. The Dam Safety Guidelines and other guidance developed by CDA can provide regulators with a basis for evaluating the safety of dams within their respective jurisdictions.

Before any water retaining structure is constructed or modified, the dam owner requires authorization according to the requirement of to the related jurisdiction and its regulatory authority, and there are often different ministries/ agencies in each province or territory that are responsible for water dams and mine tailings dams.

The regulatory framework that governs dams (water and mining) have been enforced in the Canadian provinces and territories of British Columbia, Saskatchewan, Ontario, Quebec, Labrador, the Northwest Territories, and for Dams in Canada's Nuclear Industry. Table (9) illustrates the scope of these legislations, regulations guidelines enforced in British Colombia for water supply dams, and Table (10) does the same for mining dams. These frameworks are similar in many details to those used in other territories [21].

Table 9: legislations, regulations guidelines enforced in British Colombia for water supply dams.

Legislation/ Regulation	Regulation	Guidelines other than (CDA)	~Number of Dams
Water Sustainability Act	Dam Safety-Regulation	-Inspection and Maintenance of Dams -Plan Submission Guidelines -Consequence of Failure Classification Guidelines -Compliance and Enforcement Policy -Legislative Dam Safety Review in BC-APEGBC Guidelines* - Site Characterization for Dam Foundations in BC-APEGBC* Guidelines	1814 Regulated Dams

Table10: legislations, regulations guidelines enforced in British Colombia for Mining dams.

Legislation/ Regulation	Regulation	Guidelines other than (CDA)	~Number of Dams
Mines Act	Mines Act Health, Safety & Reclamation Code	-Dam Safety Inspection Guidelines -APEGBC Professional Practice Guidelines* -Legislative Dam Safety Review in BC-APEGBC Guidelines*	118

Note: APEGBC is the Association of Professional Engineers and Geoscientists of British Columbia.

Dam safety Acts and Regulations, as established in British Colombia, serve as models for other similar Acts and Regulations in the other Canadian territories. They deal with site surveillance of dams, their formal inspections, and performing tests; by putting responsibility on the owner to assess the condition of the dam during the operation of the dam or the alteration or improvement to or replacement of the dam. This is established by, site surveillance, formal inspection of dams, carrying out of spillway gates and other mechanical components of the dam, and the electrical and communication equipment relating to the dam. The Acts and Regulations require also that these activities be carried out as frequently as specified in schedules attached to the Acts and that all results are documented in reports. Overall safety reviews are also required and reported as frequently as specified.

The regulations charge the Owner with the duty of installing and upkeep of instrumentation. The owner must install the instrumentation necessary to adequately monitor the dam and the area surrounding or adjacent to the dam, maintain or replace the instrumentation when required and ensure continuity of readings. The collected readings have to be analyzed and interpreted and reported no less frequently than is specified in the respective schedule for the classification of the dam. Moreover, an Owner who intends to install, modify, replace, or remove instrumentation relating to the dam must submit detailed plans with time schedules to a dam safety officer for acceptance.

With respect to the safety review of any dam, the Owner is expected to do this in accordance to the risk classification of dam; being, high, very high, or extreme, no less frequently than what is specified in a tables for each of these categories. The Owner has to ensure that an engineering professional who has qualifications and experience in dam safety analysis carries out this review in accordance with the requirements of the controller or a water manager.

The review shall aim to determine if the dam is safe; and, if it is determined that the dam is not safe, to stipulate what actions are required to make the dam safe. The Review Engineer has to prepares, in the form and with the content specified by the comptroller or a water manager, a report on the safety of the dam, and submit to a dam safety officer, for acceptance by the dam safety officer [22] [23].

8. Summary Points and Conclusions

1. Dam safety inspections and reviews are impotent safeguards against threats that might endanger any dam, newly built or has been in service for many years. Any anomaly in the behavior of such dam, when early discovered and then investigated and treated, saves the dam from further consequences before things become worse or out of control leading to failure. As a final result this will save the community from fatal losses and damages.
2. Dam inspections, as a first step, are to be performed, by experienced operation personnel. More comprehensive inspections and safety reviews are carried out as next preferably by professional inspection engineers and they have to be done in such frequency which is commensurate with the dam classification hazard and its size and importance, and its past behavioral history. Reference to design documents and reports of past performance become as handy tools for comprehensive safety reviews. If any sign of new adverse development appears during the operation of the dam, then dam personnel may call for an

urgent review and action to remove the source of the problem or to ease its impact. Immediate inspection and safety review may become also necessary after passing exceptional floods and earthquake events, and in such safety reviews specialists help is required.

3. Notwithstanding the outcome of such reviews, constant monitoring and observations of the dam, the appurtenant structures and the environ have to be continuously made. Follow up seepage water quantities, chemical composition of seepage water samples and piezometric readings constitute vital information on the general health of the dam in the same way as all instrumentation readings and recordings. Between safety reviews, the collected data, its summaries and analyses and photographs taken shall be compiled in a report to be submitted to the dam management with recommendations. This monitoring process shall be considered as a continuous process which shall be reported as frequently as required and at previously specified periods.
4. A good practice would be to include in the dam O&M manual instructions on the frequency and scope of the monitoring, visual inspections, testing and surveys and to provide checklist of areas of interest with these instructions. Such check list may be revised with passage of time and new items may be added as become~~s~~ necessary. Maintenance works shall be based on the outcome of the Dam Safety review reports which are the final outcome of these activates.
5. Instrumentations have been manufactured to cover, wide if not all possible problems or concerns. A well designed instrumentation system in any new dam can yield valuable information during the future life of that dam. Some more instrumentation can be also added if need arises in older dams. The objective is always to have a better overall knowledge of the dam behavior and to reduce risks, and minimizing possible threats.
6. In most countries which have large number of dams, the promulgation of laws and acts dealing with dam safety issues has become the duty of governments of these countries and recognized institutions , and this matter has been taken as one of “Public Concern”. Similarly, in these countries, guidelines have been formulated by professional bodies in these countries. Other countries can make use of these laws and regulations in a general way, but they should be tailored according to the circumstances and needs of such countries.
7. The intensive developments witnessed all over the world today, taken with the rapidly accelerating population growth, drives towards constructing more new dams. This may increase the hazards and risks to population and infrastructures; unless such dams are built with all possible safety precautions applied and reviewed. Having grater diligence in observing and reviewing the safety of existing dams, which are aging with the passage of time, can also alleviate great possible dangers in the future.

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Dam Safety: Use of Instrumentation in Dams

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Abstract

Dam safety concerns do not stop at site selection, or the design and construction stages of a dam, but continue throughout its whole life. Seeing to safety issues of any dam is done by following up its behavior through visual observation supplemented and enriched by data collection from all the devices installed on or implanted in a dam to follow its reactions to the forces and conditions in action. Analysis of the accumulated data will show the safety level and the need or, otherwise, of any remedial works. Using measuring devices to quantifying seepage conditions at any dam and correlating this with water levels' fluctuations is the first main issue that concerns dam safety. Added to this, measurement of pore water and total earth pressure in earthfill dams, temperature measurements in concrete dams, uplift and displacement measurements, and measurements of stresses and strains can all give good pictures of what is happening inside the dam and/or its foundation in both types of dams. In the following work, a summary of typical instrumentations and monitoring used in evaluating causes of common problems is given. Moreover, requirements for good instrumentation program are explained and the methods of data collection, whether manual, or use of stand-alone loggers and real-time monitoring networks are touched upon. The various devices in current use are described in more details. Actual examples of monitoring systems in existing dams are presented to show the value and importance of these systems to the safety of these dams.

Keywords: Dam safety, visual observation, seepage, pore water pressure, total earth pressure, uplift, temperature measurements, displacement measurements, stand-alone loggers, real-time monitoring networks

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1 Introduction

Instrumentation of dams is the use of devices to measure safety parameters of such dams which together with visual inspections and other measurements made at the site provide powerful tools to evaluate the performance and discover early signs of abnormal behavior. Careful examination of instrumentation data on a continuing basis may reveal a possible critical condition. Conversely, instrumentation may be a mean of assuring that an observed condition is not serious and does not require immediate remedial measures.

At the design stage, many factors and parameters are assumed with respect to materials used, dam performance under the predicted loads, foundation conditions and the behavior of these foundation during operation. In most of the cases the materials are tested in laboratories and designs are based on calculations that the dam and foundation will act within the range of assumptions made fulfilling the required factors of safety.

In actual conditions, there are always large variations in materials properties, construction methods and control procedures; all leading to have conditions deferring from the original assumptions. Similarly, some loading conditions may be beyond those assumed due to gaps in recorded data or due to unprecedented or unexpected conditions. Inadequate knowledge of foundation, resulting from insufficient geological investigations or wrong interpretation and unsuitable treatment ends in undesirable safety problems.

Accumulated experience with dams coupled with increased knowledge of rock and soil mechanics and concrete technology in addition to lessons derived from many dam failures have led to the advent and progress of instrumental monitoring and the technological innovations in this field. Measurements obtained from instruments now may be electronic or made by using a mechanical device, and telemetering made it possible to relay real time measurements to remote control centers.

Although simple devices for water level measurements and seepage measurements have been in long use, the great leap was made in the United States in the 1930s; the Bureau of Reclamation began to use controlled measurements to correlate the performance of completed dam structures with design assumptions and construction practices. Construction of new dams from the 1930s until the 1970s and after came with extensive utilization of dam instrumentation and monitoring programs. Such programs have undergone great changes in recent years; because of progress in microcomputer technology. However, although many advances have occurred in data acquisition, processing, instrumentation components and measuring techniques the factors affecting dam performance or the performance indicators have not changed putting more stress on the importance of instrumentation monitoring as one safeguard for dam safety.

2. Scope of Instruments Monitoring

Instruments measurements are complementary to visual inspections which are carried out routinely for checking dam safety issues. The benefits of instrument monitoring include the following:

i) Instrumented monitoring products are quantitative data which supplement visual observations,
ii) The accuracy and sensitivity of the monitoring is defined as the instrumentation program is designed which is dependent on dams' hazards classification,
iii) Certain instruments give information on foundation conditions and within the interior of structures which cannot be followed by visual inspections,
iv) May provide long term, consistent records of data, while other methods of observation tend to provide short-term discrete records,
v) Allows the detection of subtle trends in the dams' performance under different loading conditions that may develop slowly over time,
vi) The collection of instrumentation data can be automated which allows for monitoring on a more frequent (near real time) basis,
vii) Automated monitoring systems can be used to initiate alarms for notifying dam monitoring personnel of sudden changes in the instrument data values due to normal or extreme loading conditions such as earthquakes and flooding.

All installed instruments require; however, constant attention to insure their good operation and prevents malfunctioning and giving false alarms. Typical instruments are used for checking certain areas of concern or performance aspects which generally include the following:

viii) Seepage flows,
ix) Seepage water turbidity, its salt content and temperature,
x) Piezometric levels,
xi) Water levels,
xii) Deformations or movements,
xiii) Pressures,
xiv) Loading conditions,
xv) Temperature variations within dams' body, and,
xvi) Accelerations experienced by the dam during an earthquake.

Some types of the instruments used are common for both concrete and earthfill dams; while others are restricted to only one type of them depending on the nature and type of the dam and the phenomenon to be observed and followed. Dam safety concerns to be checked by visual inspections and instrument measurement are given in Table (1) [1].

Table 1: Typical instrumentations and Monitoring used in Evaluating causes of Common

Problems/ Concerns

Problem/ Concern	**Typical Follow Up Method**
Seepage or leakage	Visual observation, weirs, flowmeters, flumes, calibrated containers, observation wells, piezometers
Boils or piping	Visual observation, piezometers, weirs
Uplift pressure, pore pressure, or phreatic surface	Visual observation, porewater pressure cells, observation wells, piezometers
Drain function or adequacy	Visual observation, pressure and flow measurements, piezometers
Erosion, scour or sedimentation	Visual observation, sounding, underwater inspection, photogrammetric survey
Dissolution of foundation strata	Water Quality test
Total or surface movement (translation, rotation)	Visual observation, precise position and level surveys, plum measurements, tiltmeters
Internal movement or deformation in embankments	Settlement plates, cross- arm devices, fluid leveling devices, pneumatic settlement sensors, vibrating wire settlement sensors, mechanical and electrical sounding devices, inclinometers, extensometers, shear strips
Internal movement or deformation in concrete structures	Plumb lines, tiltmeters, inclinometers, extensometers, joint meters, calibrated tapes
Foundation or abutment movement	Visual observation, precise surveys, inclinometers, extensometers, piezometers
Poor quality rock foundation or abutment	Visual observation, pressure and flow measurements, piezometers, precise surveys, extensometers, inclinometers
Slope Stability	Visual observation, precise surveys, inclinometers, observation wells, piezometers, shear strips
Joint or crack movement	Crack meters, reference points, plaster, or grout

	patches
Stresses or strains	Earth pressure cells, stress meters, strain meters, over coring
Seismic loading	Accelerographs
Relaxation of post- tension anchors	Jacking tests, load cells, extensometers, fiber- optic cable
Concrete deterioration	Visual observation, loss of section surveys, laboratory, and petrographic analyses
Concrete growth	Visual observation, precise position survey, plumb measurements, tiltmeters, plumb lines, inclinometers, extensometers, joint meters, calibrated tapes, petrographic analyses
Steel deterioration	Visual observation, sonic thickness measurements, tests coupons

3. Requirements for good instrumentation program

Instrumentation devices are used to supplement visual inspections in evaluating the performance and safety of dams. Any good instrumentation program should fulfill the following requirements if it is to have any meaning and to serve its purpose:

i) Instrumentation adequacy.

Instrumentation to monitor structural and functional performance should be installed in dams where complex or unusual site conditions have been encountered or where there is a high probability that failure could result in loss of life or extensive property damages. The adequacy of any instrumentation program is directly related to the size of the dam and its risk hazard classification. The installed instrumentation should be assessed from time to time by specialists to determine if it is sufficient to help evaluate the performance of the dam. When required, additional instrumentation should be installed to confirm suspicious trends or to explore an indicated potential adverse trend. At the same time instruments should be examined periodically for proper functioning.

ii) Observation of monitoring devices.

The instrumentation data should be collected by trained personnel who can recognize signs of problems and report them immediately to the higher technical level for further action. Observation data should be properly tabulated for record purposes or for immediate action if needed.

iii) Frequency of monitoring

Frequency of observation is normally governed by the following factors:

- Risk hazard classification of the dam under consideration. This is related to the dam height, impoundment size, the intensity of occupancy of the downstream and land use there,
- Seismic risk at the site,
- Age or condition of the dam,
- Frequency and amount of water level fluctuation in the reservoir; and
- History of problems or abnormal behavior at the dam.

Moreover, the frequency of instrument readings should be established at the time the instrumentation system is designed, which in its turn follows applicable regulatory requirements and other factors. Instructions to such frequency is to be specified in the O & M report of the project. But, any such specifications; however, may be changed by the dam safety inspection authority if unpredicted anomalies arise during operation.

Whenever necessary, more frequent readings, as often as hourly, daily, or weekly should be taken to monitor a suspected rapidly changing adverse condition. The frequency or number of readings may be reduced after the project has been in operation for an extended time and if performance observation data indicates that readings have stabilized.

iv) Data Analysis.

It is essential that instrumentation data be processed, reviewed, and assessed in a timely manner by specialists familiar with the design, construction, and operation of the project. Operation manuals and design information should be referred to in the evaluation of possible adverse trends. The performance observation data should be periodically analyzed to determine whether project structures are reacting as assumed in the design, and to detect behavioral conditions that may indicate the need for corrective action [2] and [3].

4. Instruments data collection methods

Instrument data collection nowadays can be done in three well established methods depending on dam's importance, complexity, and amount of observations needed in addition to available resources. The following is a brief discussion of the three general types of data collection currently in use.

i) Manual Data Collection

Manual collection methods include the use of weighted tapes, scales, calipers, survey rods, weirs, and other measurement devices which may be read manually. The measurements might also be made using electronic sensors with readout devices. These readout devices sometimes allow for digital storage of the readings which can be later downloaded to a computer in the office. In the office, the data may be graphed or tabulated for analysis and presentation. In addition, visual observations made during the data collection must be stored in a manner that allows for future reference and retrieval.

ii) Use of Stand-Alone Data Logging

Sometimes, readings are required to be collected at short time intervals, e.g. every 15 minutes, hourly or daily to meet monitoring objectives. Stand-alone data loggers are able to record data to built-in memory without a connected PC. In many cases a PC may be necessary to provide initial programming, but afterward the data logger operates independently. When recording stops, data recorded by stand-alone data loggers can be uploaded to a computer for analysis.

These recordings can be very helpful in developing an understanding of how the dam responds to changing loading conditions such as reservoir level, rainfall, and air temperature. Readings from multiple instruments can be collected simultaneously so that different parameters can be directly compared. This can also be accomplished with manual readings, but depending on the size of the project, number of instruments, location of instruments, and frequency of readings, it is generally too labor intensive if readings are needed at an interval of more than once a day. Data loggers may also be a good alternative to manual readings for remote sites or for areas of difficult access, such as a dam gallery.

There are many different types of stand- alone data loggers manufactured nowadays that are battery- or solar-powered and environmentally hardened for direct field deployment. These units can be configured using a computer or handheld device and then left in the field for unattended data collection. Also, regular visual observations should continue to be performed even if data is being collected frequently, as visual observations can often identify developing problems before an instrument registers a response.

If irregular readings are noted upon uploading the data, these should be immediately investigated, explained, and/or repeated. Also, periodic manual readings should be taken to verify the digital readings.

iii) Real-time Monitoring Networks

If both frequent unattended data collection and real-time display or notifications are required, then an automated data acquisition system (ADAS) may be the best option. Automated systems can also save labor and reduce the time for data evaluation by providing automated data retrieval from a remote location. The data is typically retrieved from the site periodically and automatically loaded into a database for presentation to the end user. Using programmable ADAS equipment data can be processed into engineering units, evaluated for alarm conditions, and displayed in real-time to operations and dam safety personnel. These displays can be customized to present the monitoring results in the format needed to make decisions. For operation personnel this may mean simple displays that show normal or alarm conditions. The interface for the dam safety personnel usually warrants a more comprehensive presentation for evaluation of short term trends, correlation relationships, alarm thresholds, statistical parameters, and geographic relationships. Although ADAS provides remote monitoring of the project, regular site visits are still required to perform the scheduled visual observations and system maintenance tasks.

There are two general system configurations that are used to automate the collection of performance monitoring data on dams:

a) Host-driven systems,

The host-driven system consists of a central intelligent host (master) device and remote units (slaves) that are pooled by the master unit to collect the instrument readings.
Because the intelligence is primarily in the host device, the system performance relies heavily on maintaining stable uninterrupted communications. Examples of host driven systems are supervisory control and data acquisition (SCADA) systems and PC based systems.

b) Node-driven systems

This system is characterized by putting the intelligence at each node in the network. A node would be a location on the dam site that monitors a single instrument or a group of instruments but is physically separated from the other nodes. Each node is capable of stand- alone operation and can be programmed to collect data and make alarm notification decisions on its own.

Communications between the nodes can be accomplished by wide variety of wireless and hardwire methods. The best method will depend on site conditions, communication services available, and the real-time monitoring and notification needs of the project. The node-driven architecture is more commonly used for dam safety monitoring; because the instrumentation tends to be widely distributed around the project site and in locations where power is not readily available. If a node detects an alarm condition, then it can immediately communicate with the other nodes, but under normal conditions, communications are kept to a minimum. A properly designed node-driven system can also provide improved reliability. In the event of the loss of communications or equipment damage in the network, the other nodes will continue to function independently. For critical systems, it is also desirable to have multiple communication paths that can be utilized. The primary advantage of an ADAS is to allow for the near real-time collection and reduction of the instrumentation data so that dam operators and dam safety personnel can rapidly evaluate the conditions at the dam. A properly designed ADAS provides real-time remote notification of a significant change in the performance or conditions at the dam 24 hours a day, 7 days a week.

Field implementation of real time monitoring system and establishment of observatory for such purpose may be done by; the required sensors will be procured and deployed at the site nodes. Sensors having proper compatibility with the data acquisition systems and with adequate sensitivity and resolution will be selected. Also, sensors network with optimum spatial coverage, both on the ground and in the subsurface will be taken into consideration.

In a modern observation network information such as; actual displacements/ movements at different locations, stress, pore-water pressure, water seepage, rainfall and seismic parameters that may affect the stability of the embankment is being targeted. This is being done by combination of surface and sub-surface mounted monitoring sensors. Sub-surface sensors will include biaxial in-place inclinometers to be installed at different depths in the interfacial zones of different materials down the borehole to measure the sub-surface displacements/ movements, in addition to piezometers in the boreholes to measure the variation in pore water pressure. Also, borehole seismometers can be deployed to measure the seismic parameters in case of an earthquake. Depths of these sensors will be decided on the basis of borehole geological logging information.

All these surface and sub-surface sensors will be connected through wire to the specific nodes on the surface placed in close proximity to the sensors/boreholes. These nodes will communicate wirelessly with the gateway (i.e., automatic data acquisition system) placed in the site observatory. The gateway will store the data from different nodes of all the sensors and will upload the data on real time to the cloud for further processing through cloud-based software.

The proposed modern framework and architecture of wireless instrumentation and real-time monitoring scheme for earth dam are shown in Figures (1 and 2) [4].

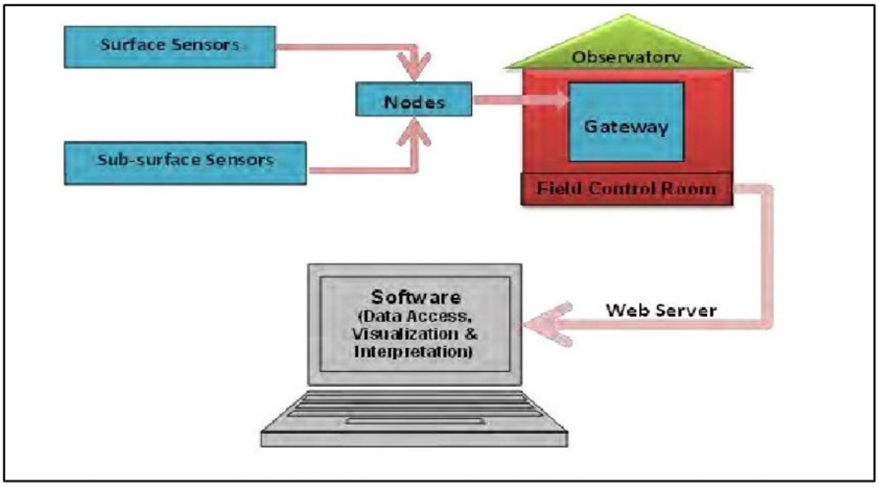


Figure 1: Proposed simplified framework for wireless instrumentation and real-time monitoring of earthfill dam site

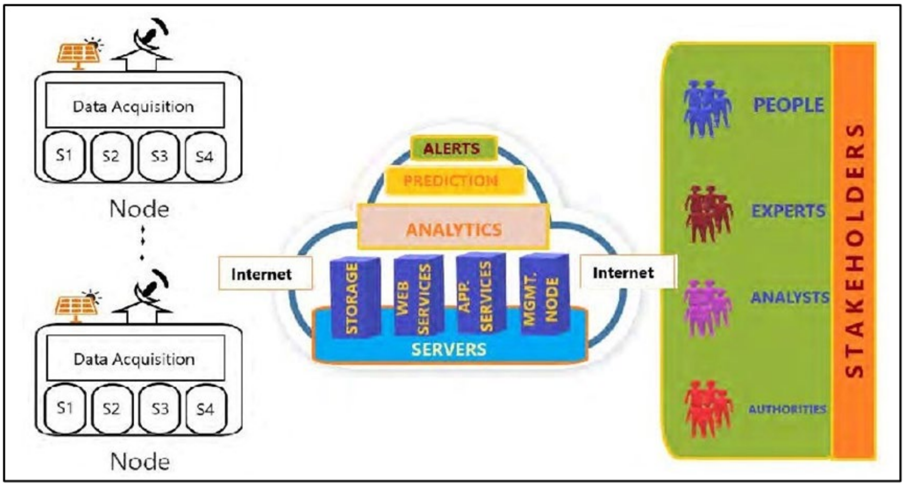


Figure 2: Architecture of the Proposed wireless instrumentation and real- time monitoring model where S1, S2, S3, are the different sensors integrated into IoT Nodes supported by solar power

A properly designed and installed instrumentation and automated real time data acquisition system can provide cost effective and reliable information for both long term monitoring and assured safety aspects. Similar instrumentation and monitoring system also can be used for the dam abutments and reservoir rim.

Dam safety instrumentation monitoring challenges, whether for a fairly straightforward problems; such as organizing historical data or a more complex issue like synchronization of multiple automated and manually read data sources, can be surmounted. Advances in automated data collection and development of software for integrating geo-monitoring data offer solutions to these and many other instrumentation challenges.

One example of using a combination of automated and manually read instrument that are synchronized into a single data management database platform is the OPC’s Rocky Mountain Hydroelectric Plant surveillance and monitoring plan. Integrated data management software allows OPC to quickly access charts and reports, set alarm notifications, and use templates to easily generate instrumentation reports [5].

The **Rocky Mountain Hydroelectric Plant** is a pumped-storage power plant located 10 miles northwest of Rome in the U.S., State of Georgia. It is named after Rocky Mountain on top of which the plant’s upper reservoir is located and it has an installed capacity of 1,095 megawatts, Figure (3) [6].

Figure 3: Rocky Mountain Hydroelectric Plant is a pumped-storage power plant

The advantages and disadvantages of the various data collection methods are summarized and compared in Table (2) [7].

level

Table 2: Summary of Data Collection Methods, Advantages and Disadvantages

Data Collection Method	Advantages	Disadvantages
Manual Data Collection	• Generally simple to perform and do not require high level of expertise. • Personnel are already on site for regular visual observations. • Data quality can be evaluated as it is collected.	• Labor intensive for data collection and reduction. • Not practical to collect frequent data. • Potential for errors in transporting\data from field sheets into management\ presentations tools. • May be impractical for remote sites where personnel are not frequently on site
Stand- alone Datalogging	• Frequent and event driven data collection. • Consistent data collection and electronic data handling. • Equipment is fairly inexpensive and simple to set up.	• Require some expertise to configure data loggers. • Data quality cannot be evaluated until it is collected from the field. • Potential for lightning strikes. • Power source needs to be considered.
Real – time Monitoring Networks	• Frequent and event driven data collection. • Consistent data collection and electronic data handling.	• Automation may encourage complacency if overall monitoring program is not well defined or understood.

	• Real-time display and notifications (24\7). • Can remotely change the monitoring frequency and data collection configurations as needed. • Allows for rapid evaluation of monitoring results.	• Requires a higher level of expertise to install and maintain. • Higher cost of installation and periodic maintenance. • The importance of frequent routine visual inspections may be overlooked or discontinued somewhat due to the real- time presentation of readings. • Potential for lightning strikes. • Power source need to be considered.

5. Types of measurements, their significance and used Instruments

Types of instruments needed to follow changing conditions of any dam normally fall into many categories according to type of measurement performed. These categories are; water levels and water pressures measurements, seepage and leakage measurements, movement measurements, stress and strain measurements in addition to temperature and, seismic monitoring and measurements. The type of measurements, devices used, and methods applied are described in the following:

5.1 Water level and water pressure measurements

Reservoir and tail water elevations should be measured to provide a continuous historical record of these variables. Water levels in the reservoir and in downstream waterways have a direct influence on the quantities of seepage and leakage through a dam and its foundations and on uplift pressure on dam base. When the hydraulic head (the difference between upstream and downstream water levels) changes, then it may result in a significant change in seepage and leakage rates, but sudden changes in seepage quantities may indicate; however, a genuine problem in the foundation which needs to be investigated at once. This was proved in Mosul Dam (Iraq) where dissolution of gypsum in the foundation could be related to piezometric readings directly influenced by the upstream and downstream water levels. Such abrupt changes of the hydraulic head on the foundation was taken as indication of the opening of new dissolution seepage paths and formation of new cavities that required immediate injection of grouting materials [8].

Water levels and water pressures are interrelated. Measuring water level in the reservoir on a regular basis and recording this level at the time of each dam safety inspection is needed for several reasons. Recording water elevations in downstream tail water elevations is also important. These elevation data are used to interpret other instrumentation measurements, including the determination of uplift water pressure’s distribution beneath the dam. Uplift pressures and seepage rates are directly dependent upon the difference between reservoir and tail water elevations.

Water levels may be measured by simple elevation gauges, such as fixed staff gauges, free standing, or fixed on permanent structures in the reservoir, Figures (4 and 5) [9] and [10].

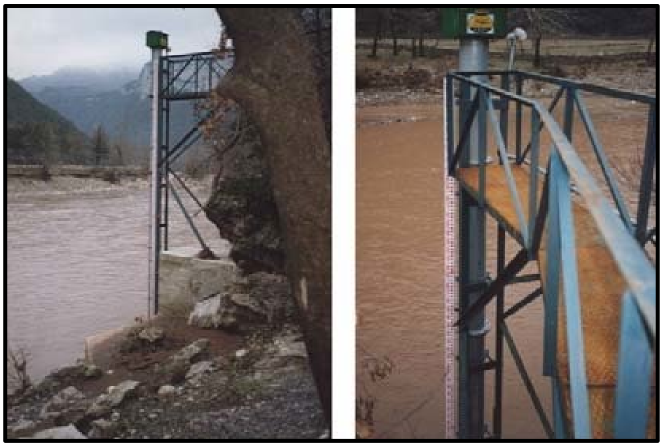


Figure 4: Water level staff gauge at Kovali Dam, and at Sultan reedfields, Kayseri. In Turkey [9]

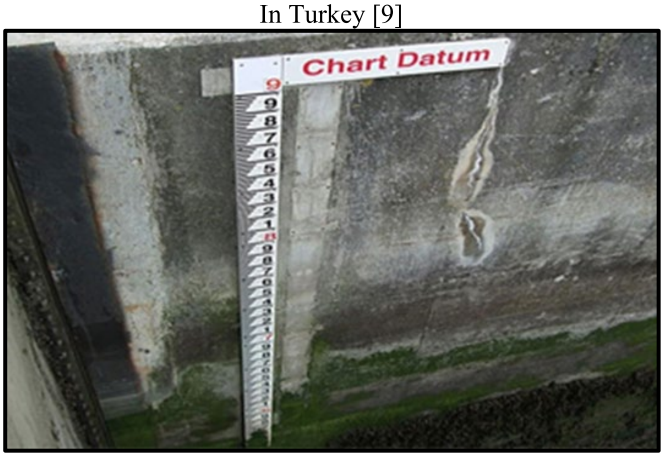


Figure5: Staff Gauge fixed to Spillway Pier [10]

Measurements may be done also with more complex water-level sensing devices. For automated monitoring, a float and recorder, ultrasonic sensor, bubbler, or other similar instruments are used.

Observation wells and Piezometers can be used to measure water levels in dam's body and surroundings. They are usually vertical pipes with a slotted section at the bottom or a tube with a porous tip at the bottom. They are typically installed in boreholes with a seal at the surface to prevent surface water from entering the borehole. The depth to the water level is measured by lowering an electronic probe or weighted tape into the pipe, Figure (6a). Observation wells are appropriate only in a uniform, pervious material. In a stratified material, observation wells create a hydraulic connection between strata. As a result, the water level in the well is an ambiguous combination of the water pressure and permeability in all strata penetrated by the borehole. Such observation data may lead to erroneous conclusions regarding actual water pressures within the dam and foundation. One variation to such wells to avoid this complication is to use piezometers of the type shown in Figure (6b) where perforated zones can be used in the layers to be tapped and sealing materials are fixed to contain these layers between them [11].

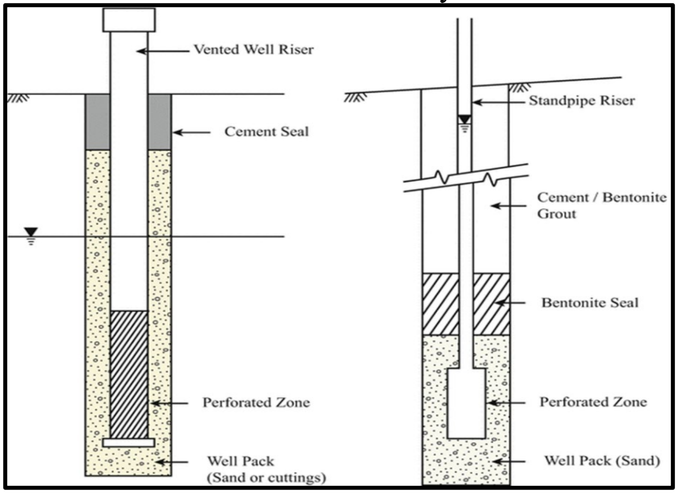


Figure 6a: to the left: Simple Observation well. Figure 6b:to the right is a Piezometer [Modified after 11]

Another type is the twin-tube hydraulic piezometers, sometimes referred to as the Casagrande-type piezometers Figure (7). This type is used to detect, measure, and monitor water pressure in permeable soil or rock specifically at the installed depth of the measuring tip. Typically, a bentonite seal is installed immediately above and sometimes below the filter. One pair of tubes is installed where water pressure measurement with a vented filter is required to provide water inlet and outlet for internal flushing to clean the filter and to remove trapped air. The tubes are extended more or less horizontally in trenches through the fill or foundation to a readout unit. The water level can be read by portable acoustic water level meter or automatically with a pressure transducer inserted in the standpipe or connected to the filter to form a closed-circuit piezometer [12].

The sensing zone (screened length or porous tip) of an observation wells and open standpipe piezometers is susceptible to clogging, which can increase lag time or result in failure of the instrument. This susceptibility can be diminished by a properly designed filter pack that meets filter criteria with the surrounding soil and properly sized perforations that are compatible with the filter pack. Open standpipe piezometers are the standard against which all other piezometers are judged. They are simple, reliable, inexpensive, and easy to monitor. The main shortcoming of open standpipe piezometers is the need for a discharge of water to or from the tube for it to adapt to pore pressure changes. Because of this, there is a noticeable time lag in recording the pores pressure changes if they occur quickly. Other types of Piezometers are the Pneumatic Piezometers, and the vibrating wire type [11].

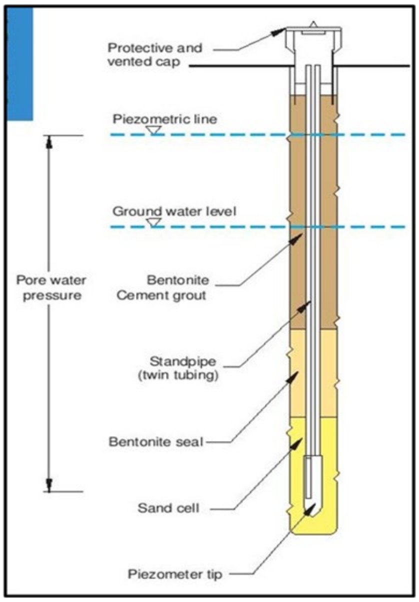


Figure 7: Typical installation of twin-pipe Casagrande type Piezometer [12]

Pneumatic piezometers consist of a porous filter connected to two tubes which have a flexible diaphragm between. The diaphragm is held closed by the external water pressure. The end of one of the tubes is attached to a dry air supply and a pressure gage. Air pressure is applied until it exceeds the external water pressure acting on the diaphragm, which deflects the diaphragm and allows the air to vent through the other tube. The air supply is shut off, and the external water pressure and internal pressure on the diaphragm equalize allowing the diaphragm to close. The residual internal air pressure is taken as the external water pressure. Alternatively, the water pressure can be taken as the air pressure required to maintain a constant flow through the tubes. Some constant flow types use a third tube connected to a pressure gage to measure pressure at the diaphragm rather than at the inlet to reduce potential errors and eliminate the need for individual calibration curves.

Typical pneumatic piezometer is shown in Figure (8), and a typical vibrating-wire strain gage piezometer is illustrated in Figure (9).

In Figure (9), a prestressed wire is attached to the midpoint of the diaphragm such that diaphragm deflections cause changes in wire tension. Consequently, a change in pore pressure deflects the diaphragm and changes the tension in the wire. The tension is measured by plucking the wire, using an electric coil, and measuring the frequency of its vibration. The wire vibrates in the magnetic field of a permanent magnet causing an alternating voltage to be induced inside the plucking coil. The frequency of the output voltage is identical to the frequency of the wire vibration and is transmitted to a frequency-counting device. A calibration curve or table is then used to calculate the pore pressure from the measured frequency change.

Another variation of the electrical pore pressure cell piezometers is the bonded resistance strain gage piezometer, Figure (10). It has an electrical pressure transducer with a diaphragm in direct contact with pore water. Strains induced in the diaphragm by the pore pressure are sensed by strain gages, bonded to the diaphragm. Hence, the transducer output signal can be used as a direct measure of pore pressure.

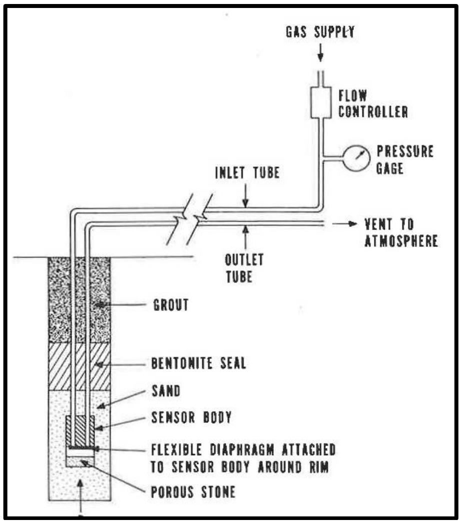


Figure 8: Typical pneumatic piezometer [12].

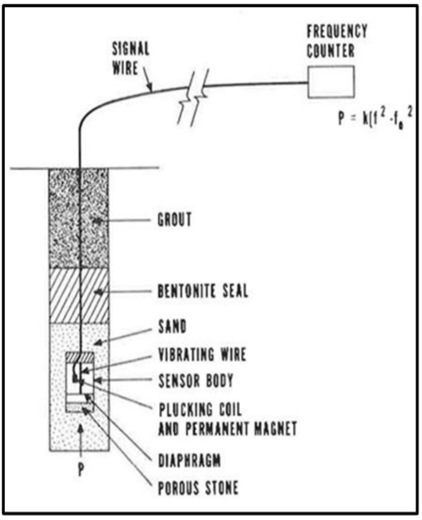


Figure 9: Vibrating wire strain gage piezometer [12].

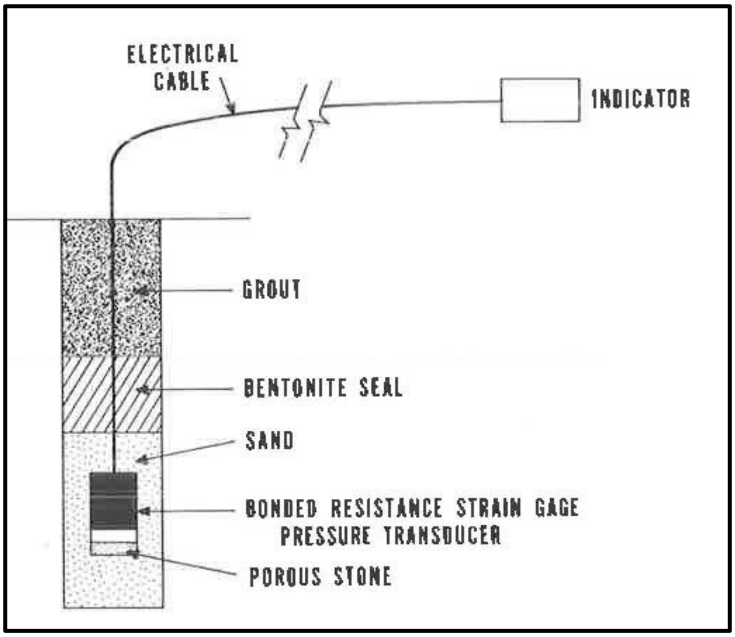


Figure10: Bonded resistance strain gauge piezometer12]

Electrical pore water pressure cells are used also for the same purpose; such as the vibrating-wire strain gage piezometer. Vibrating wire piezometers consist of a porous stone connected to a sealed metal chamber with a diaphragm adjacent to the stone. The diaphragm separates the pore water from the measuring system.

Typical electric pore water pressure types of piezometers are shown in in Figure (11).

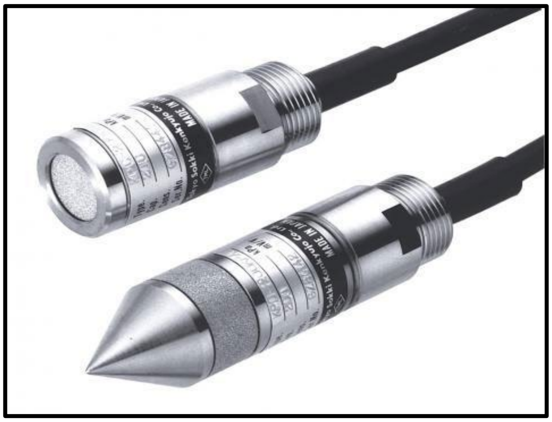


Figure 11: Typical pore pressure cells for electric type piezometers

Time Lag when pore water pressure changes i. e. the time required for water to flow to or from piezometers to effect equalization is called the hydrostatic time lag. It is dependent primarily on the permeability of the soil, the type and dimensions of the piezometer, and the magnitude of change in pore water pressure. Open standpipe piezometers have a much greater hydrostatic time lag than diaphragm piezometers because a greater movement of pore water is involved. The term "slow response time" is used to describe a long hydrostatic time lag [13].

5.2 Seepage and leakage measurement

Water-retention ability of any dam takes its importance not only from saving valuable stored water, but also from observing any threats to the stability of the dam caused by seepage or leakage through it or through its foundation. Seepage from a reservoir is the interstitial movement of water through a dam, the foundation, or the abutments. It is different from leakage, which is the flow of water through caverns and/ or cracks. Seepage and leakage through a dam should not be large enough to erode material from inside the dam body. Such internal erosion can cause undermining or piping in embankment dams, and loss of material strength or density in concrete and masonry dams.

Seepage and leakage are commonly measured with weirs, Parshall flumes, and calibrated containers. Other types of flow measuring devices such as flow meters may be appropriate in special circumstances. Geophysical surveys can be used to determine flow path and direction [14], [15] and [16].

Quantity of seepage and/ or leakage from dam body or its foundation is directly proportional to the head difference on the dam and to the permeability of the material of the dam or the foundation. Variation of head difference is usual in the normal reservoir water fluctuation during operation which causes subsequent changes in seepage or leakage quantities. Any change in seepage and/ or leakage volume not related to reservoir level must be evaluated immediately and significant or rapid changes in seepage and leakage related to the reservoir level should also be investigated. An increase in seepage and/ or leakage may be an indication of piping. A decrease in seepage and/ or leakage may indicate clogged drains. A decrease in seepage may also indicate that seepage is increasing at a location other than that being measured, which could lead to piping. Cloudy or turbid seepage water can indicate piping. New seeps and/ or leaks may also be indications of developing problems.

Another variable that affects the amount of seepage and/ or leakage is the development of the steady-state phreatic surface in a newly constructed earthfill dam.

The steady-state phreatic surface development can take years, during which a gradual increase in seepage and/ or leakage may occur. For dams on soluble rock foundations such as gypsum, karstified limestone or halite, seepage may increase with the progress of time due to dissolution of the rock. In such cases, steady increase in seepage may indicate developing problems, such problem was experience in Mosul Dam Project and represented severe threat to the stability of the Dam [8].

Water quality measurements can provide data on the source of seepage, or piping and on the dissolution of foundation rock. Common water quality measurements include; the pH value, temperature, conductivity, total dissolved solids, total suspended solids, and the variety of minerals present in seepage water such as sodium, potassium, carbonate, bicarbonate, sulfate, and chloride.

Devices used for seepage and leakage quantity measurements are weirs, Parshall flumes, calibrated containers. These are fairly standard in design and do not need much elaboration here. The reader may be referred to an excellent publication prepared by the United State Bureau of Reclamation in collaboration with the United Department of Agriculture for more details on these devices [17].

Dam safety investigation and periodic monitoring can also be done by variety of geophysical methods to establish source of any anomaly in dam's behavior such as sudden increase in seepage quantity. Such methods can be performed by a many methods such as electromagnetic profiling, electrical resistivity tomography, self-potential method, use of ground penetration radar, seismic methods, temperature methods and Micro-Gravimetry [18].

5.3 Movement measurements

When evaluating the performance and safety of a dam, it is essential that movements be carefully watched. This is especially true for concrete and masonry dams where even small shifts in position can lead to grave consequences. Measurements of movements in concrete dams are made with several different instruments including plumb lines, inclinometers, extensometers, strain meters, and joint meters.

Instruments for the measurements of concrete dams properties including movements are given in Table (3) [19] and [20].

Table 3: Concrete dam instruments and measurements (ASCE Guidelines) [20]

Property Measured	**Measurement Location**	**Typical Instrument**
Alignment	Crest, other locations of interest	Total station, laser, geodimeter
Rotation	Within Concrete	Inclinometer
	Within foundation	Tiltmeter
Differential Movement	Across joint or crack	Strain gauge, extensometer, joint meter, crack meter
	Within foundation	Extensometer, tiltmeter
Water Pressure	Uplift across base	Piezometer, observation well
	Within concrete	Piezometer
	At drains	Pressure gauge
Stress and Strain	Foundation	Total pressure cell, load cell, strain meter, flat jack
	Within concrete	Total pressure cell, strain meter
Internal Temperature	Within concrete	Thermocouple, RTD, thermistor
Seepage Quantity	Any location of interest	Calibrated container, weir, flume, flow meter
Seepage Quality	Any location of interest	Turbidity meter
Anchor Load Retention	Anchor head	Load cell, jack and pressure gauge,
Earthquake Response	Crest, free field or other location of interest	Strong motion accelerometer

In embankment dams, the key movements to monitor include foundation and embankment settlement, and vertical and horizontal deformations within the embankment and its foundation. Embankment movement measuring instruments include settlement sensors, foundation baseplates, inclinometers, extensometers, surface points, and various survey instruments.

Movement can be divided into three types;

i) Surface movement: Which is the horizontal or vertical movement of a point on the surface of a structure relative to a fixed point outside the structure and it is usually determined by some type of surveying. Additionally, in concrete dams there is the rotational movement to account for;
ii) Internal movement: Which is the horizontal or vertical movement within the structure and it is usually determined relative to some point on the structure or in the foundation, and;
iii) Crack or joint movement: Which is the horizontal or vertical movement of one part of a structure relative to another part of a structure and it is usually measured across block joints or cracks in concrete structures or cracks in earth structures.
iv) Movement can occur in any direction; therefore, measurements are made in three mutually perpendicular directions which are vertical, transverse horizontal, and longitudinal horizontal directions. When movement in one or more of the three directions is found to be negligible, then measurement in those directions can be dropped.

Movements are direct result of external and/ or internal loads. Embankments settle and spread over time as the result of consolidation and secondary settlement of the dam and foundation from self-weight. Embankments also deform due to external loads produced by reservoir water, rapid drawdown, earthquakes, undermining, swelling clays, and piping. Concrete structures deform due to internal loads such as pore water pressure, cooling, and alkali aggregate reaction of concrete, in addition to external loads caused by air and reservoir temperatures, solar radiation, reservoir levels, uplift pressure, wind, earthquakes, undermining, ice, overflowing water and foundation settlement.

Movements in response to such loads are normal and acceptable, provided they are within tolerable ranges and do not cause structural distress. Embankments are less brittle than concrete structures and can undergo larger movements without distress. As result, measurements of surface movements of embankment dams are typically less precise than those for concrete structures.

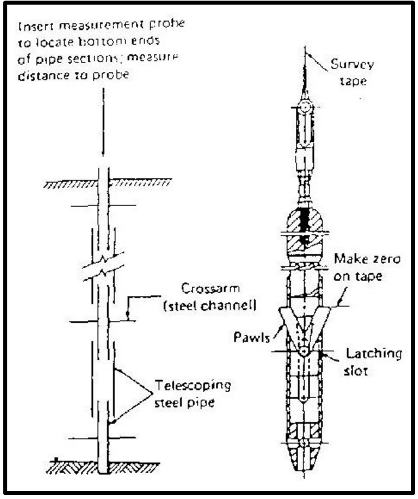
Sudden or unexpected change of direction, magnitude, or trend of surface movement could indicate developing problems in both types of dams. Therefore, internal movement and external measurements of both concrete and embankment dams and their foundations should be detailed and precise.

Surface movement is done by surveying methods. Survey points are constructed on the structure in such a way as to ensure observation of the three dimensional movements of dam parts. All observations are referred to stable bench marks fixed in the dam surrounding so that they are not subject to movement from freeze-thaw action or traffic.

Internal movement in embankments can be measured by variety of devices which are placed in embankment fill as filling proceeds. Such instruments are used for monitoring the changing distance between two or more points along a common axis without use of a movable probe. They are used for monitoring settlement, horizontal deformation, or strain. The range of such instruments includes:

i) Probe extensometers used for monitoring vertical compression of the fill or foundation of embankments and they may be of the cross arm gauge type, mechanical probe Figure (12), or induction coil gauge, Figure (13) or magnate/reed switch gauge, Figure (14).
ii) Monitoring of settlement, horizontal deformation or strain can be made also by; settlement platforms, Figure (15), buried plates, and gauges with electrical linear displacement transducers, Figure (16).
iii) Other types such as subsurface settlement points are used to measure consolidation settlement in the fill as illustrated in Figure (17).
iv) Fixed borehole extensometers may be used for monitoring the changing distance between two or more points along the axis of a borehole, without use of a movable probe and these are devices installed in boreholes in soil or rock, Figure (18).
v) Inclinometers are also installed which measure deformations parallel and normal to the axis of a flexible pipe by means of a probe passing along the pipe Figure (19) [21].

In concrete dams, many types of instruments and devices are used. For internal movement measurements these include, plumb lines, extensometers, inclinometers, tiltmeters, strain meters, joint meters and plummets. For external movement measurements, surveying is done by collimation measurements, trilateration measurements with the use of many types of instruments and devices such as theodolites, stationary instrument piers and plates, fixed targets, electronic distant measuring techniques (EDM), precise levelling techniques, in addition to surface monitoring points and tiltmeters. Full description of the methods and the devices and their installation is covered in USBR manual on the instrumentation of concrete dams manual [22].



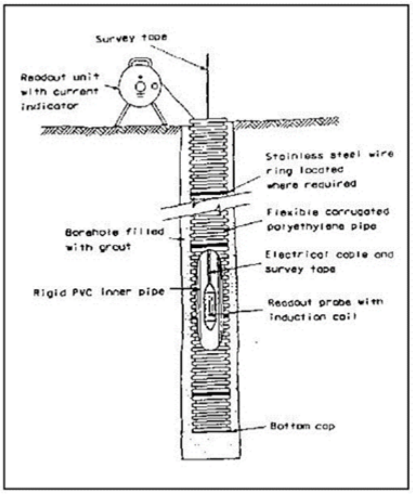
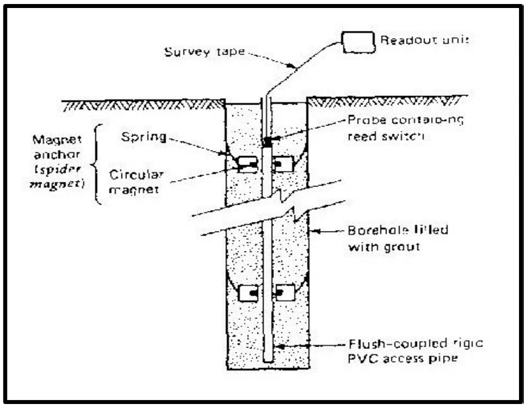

Survey tape
Readout unit with current indicator
Stainless steel wire ring located where required
Flexible corrugated polyethylene pipe
Borehole filled with grout
Electrical cable and survey tape
Rigid PVC inner pipe
Readout probe with induction coil
Bottom cap

Figure 12: (Left) Crossarm gauge: (a) Schematic of pipe arrangement, and (b) measurement probe.
Figure 13: (Right) Schematic of slope indicator extensometer probe installed in a borehole


Readout unit
Survey tape
Probe containing reed switch
Magnet anchor (spider magnet)
Spring
Circular magnet
Borehole filled with grout
Flush-coupled rigid PVC access pipe

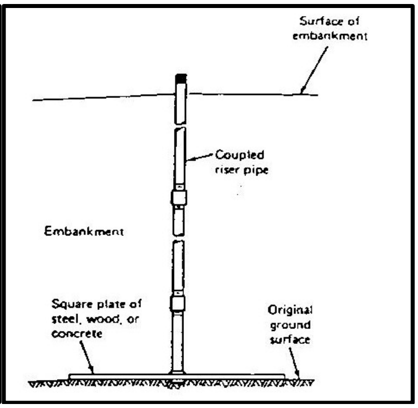
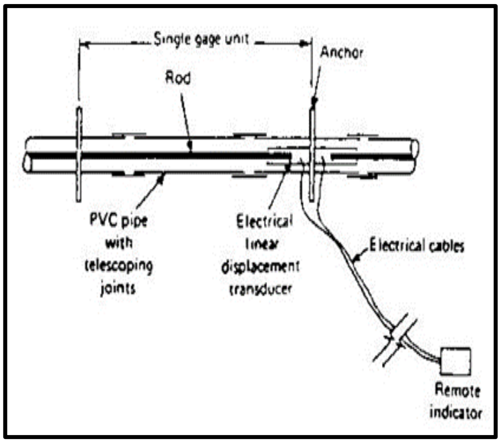


Figure 14: (left) Schematic of probe extensometer with magnet/reed switch transducer, installed in a borehole. Figure 15: (Right) Typical settlement platform.


Single gage unit
Anchor
Rod
PVC pipe with telescoping joints
Electrical linear displacement transducer
Electrical cables
Remote indicator

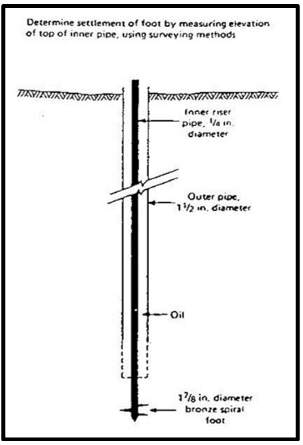
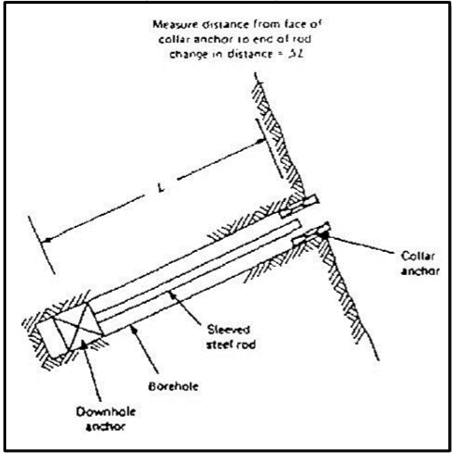

Determine settlement of foot by measuring elevation of top of inner pipe, using surveying methods
Inner riser pipe, 1/4 in. diameter
Outer pipe, 1 1/2 in. diameter
Oil
1 7/8 in. diameter bronze spiral foot

Figure 16: (left)Schematic of fixed embankment extensometer with electrical linear displacement transducer. Figure 17: (Right) Schematic of spiral foot subsurface settlement point.


Measure distance from face of collar anchor to end of rod change in distance = δL
L
Collar anchor
Sleeved steel rod
Borehole
Downhole anchor

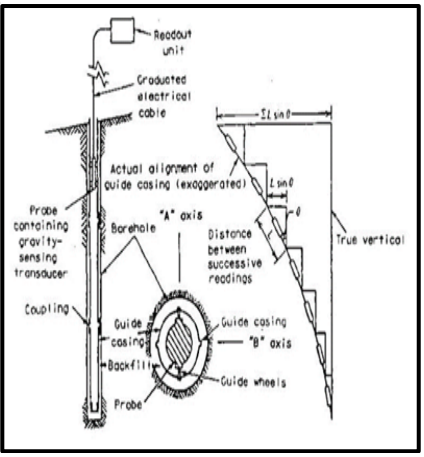

Readout unit
Graduated electrical cable
ΣL sin θ
Actual alignment of guide casing (exaggerated)
L sin θ
θ
Probe containing gravity-sensing transducer
"A" axis
Borehole
Distance between successive readings
L
True vertical
Coupling
Guide casing
Guide casing
"B" axis
Backfill
Guide wheels
Probe

Figure 18: (left) Operating principle of fixed borehole extensometer.
Figure 19: (Right) Principle of inclinometer operation.

5.4 Stress and strain measurements

Actual stresses within a dam which result from external loads may not correspond always with those calculated in the design, so it is important to find actual stresses in a completed dam to verify design assumptions and to supply data for future design improvements.

Soil stresses within dams; however, cannot be determined directly but only indirectly by measuring the pressures in the structure or the reactions between the structure and supporting elements outside the structure. Such soil pressures can be measured directly by means of pressure cells [23].

Stresses are caused by the static loads acting on the dam which are weight of the dam, and earth and water loadings. Total pressure measuring cells are used to measure the actual stresses at selected locations, such as within the dam body, between a dam and its abutments or foundation, or between certain components of the dam.

Several types of devices are used to measure the static total pressure in a dam. Two main kinds of total stress measuring devices are in use: the embedded earth pressure cells and the contact earth pressure cells. Instruments of the first type are installed within the fill to determine the distribution, magnitude and direction of total stresses within an embankment, while the second type measures total stress against structures such as walls, culverts, spillways or rock abutments.

Commercially available earth pressure cells include the following types: pneumatic, hydraulic, vibrating-wire, bonded resistance strain gage, and unbonded resistance strain gage [24]. They consist of two flexible diaphragms sealed around the periphery, with a fluid in the annular space between the diaphragms. Pressure is determined by measuring the increase in fluid pressure behind the diaphragm with pneumatic or vibrating-wire sensors. Earth pressure cells should have similar stiffness as the surrounding soil to avoid inaccurate measurements of in-situ stress caused by arching. An example of the vibrating wire type is shown in Figure (12) [25].

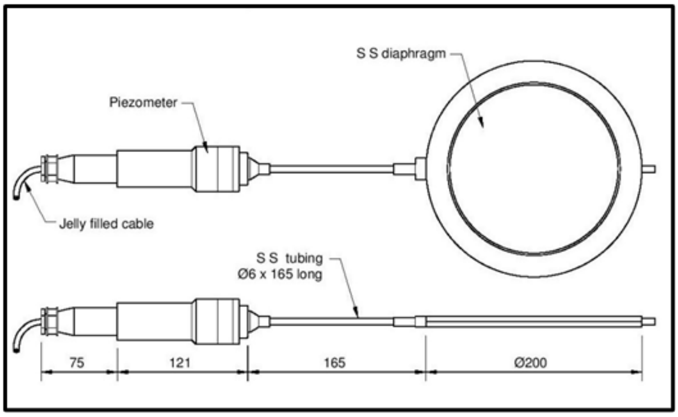


Figure 12: Earth pressure cell with cellular diaphragm [25]

Carlson-type earth pressure cells can also be used to measure pressure against structures. It consists of a chamber with a diaphragm on the end. Deflection of the diaphragm is measured by a Carlson-type transducer and is converted to stress.

For concrete dams, the modulus of elasticity, creep coefficient, and the Poisson's ratio for concrete can be determined from the laboratory testing of concrete field cylinders' cores. These values are needed to convert strain measurements to stress. Stress in concrete structures can be measured directly with total pressure cells or Carlson-type cells designed to have a stiffness similar to concrete.

A variety of mechanical and electrical strain gauges are used to measure strain in concrete structures. Some of the instruments are designed to be embedded in the dam during construction, and others are surface mounted following construction. Strain gages are often installed in groups so that the three-dimensional state of strain can be evaluated. One common instrument is the Carlson elastic wire instrument Figure (13).

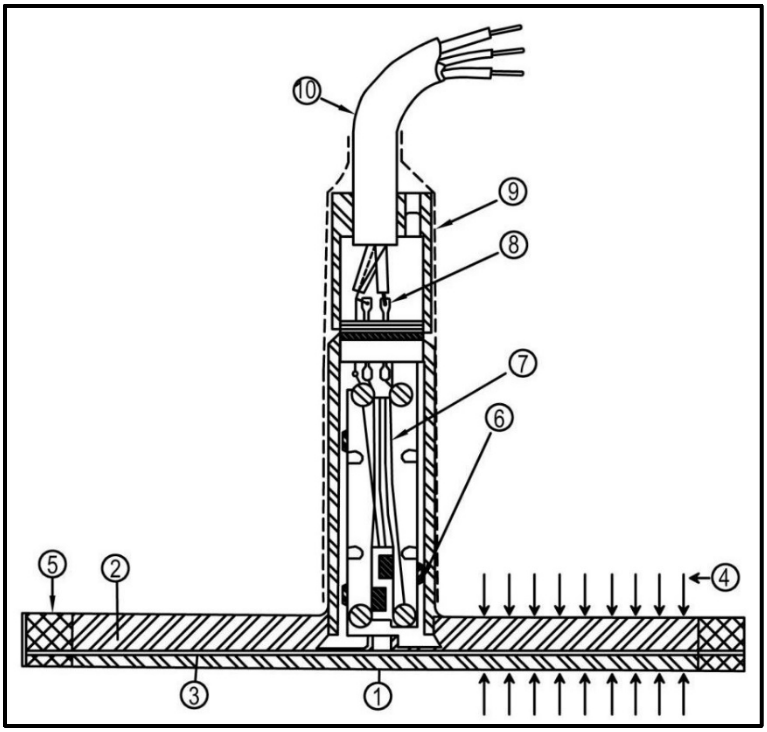


Figure 13: Carlson stress meter (after Golzé 1977). (1) internal Plate, (2) external Plate, (3) mercury film, (4) stress being measured, (5) compressible material, (6) steel bar, (7) ceramic spool, (8) glass insulated terminals, (9) fabric cover, (10) conductor cable [26].

This is a dual purpose instrument in which the main element is an elastic music wire coil. The instrument takes advantage of the fact that the electrical resistance of steel wire varies directly with the temperature. This instrument can also be used for measuring tensile stresses because the changes in the electric resistance of the wire are also in direct dependence on the tension in the wire.

The external pressure, acting on the plates, creates a corresponding pressure in the mercury, which causes deflection of the diaphragm towards the external side, by which there comes about an activation of the device for measuring strains. This consists of two threads of steel wire, placed on sheaves (pulleys), connected by a steel frame. The deflection of the diaphragm tensions the wire from one of the threads while loosening the other, in equal proportion. This instrument makes use of the fact that an increase in tension increases the frequency of vibration of the wire when plucked. The frequency is measured by use of a magnetic circuit. The measurements can be made by electrical readouts.

5.5 Temperature measurements

No temperature measurements are recommended for embankment dams. Proposed concrete gravity and arch dams should have an array of instruments to measure internal and surface temperatures along a transverse plane through the maximum section. In addition, concrete arch dams should have a string of instruments to measure reservoir temperature along the height of the maximum section Figures (13,14) [27].

Temperature measurements of a dam, foundation, ambient conditions are used to reduce data from other instruments, increase precision, or to interpret results. For example, movements of concrete dams and changes in leakage in concrete dams are often related to changes in temperature.

Temperature is also measured in concrete dams under construction to evaluate mix design, placement rates, and block and lift sizes; to time grouting of block joints; and to assess thermal loads.

During the operation of concrete dam, warm sunlight on the downstream face can create strain that may result in differential stress between the downstream face and the cooler upstream face that is mostly below water. Conversely, during winter the relatively warm water on the upstream face may cause differential stresses versus the downstream face that may be exposed to very cold air temperature.

Resistance thermometers or thermocouples can measure the temperatures of a dam, its foundation, and other instruments.

Resistant thermometers operate on the principle of changing resistant in a copper wire as the temperature changes. The instrument is read with a Whatstone bridge type read out box. Figure (15) shows typical resistant thermometers used in USBR dams.

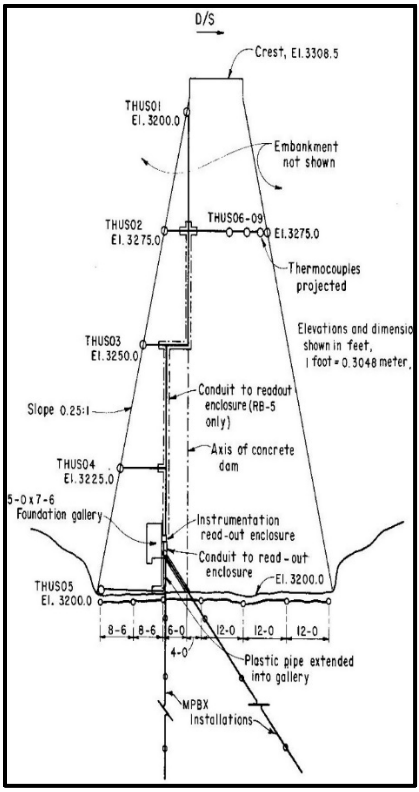
5.6 Seismic loads instrumentation

The main reason for seismic instrumentation of dams is monitoring their behavior when an earthquake occurs. Seismic instrumentation is used to record the responses of a structure, foundation, and abutments to seismic events. One of the specific applications of the acquired measurements is to furnish data to decide if the structure will continue to function as intended, and for the purpose of evaluating its behavior in such an event verifying design assumptions made for such case.

Major vibrations at a dam site could cause liquefaction of the foundation soils or the embankment, resulting in potential stability problems or the cracking of concrete slabs or other structures. The measurement of earthquake motion can help improve the designs of future dams, so that they can better resist earthquake effects, and help assess damage after significant earthquake occurrences. Because of the inability to predict exactly when or where earthquakes will occur, it is desirable to install instruments for most structures in areas of high earthquake incidence.

The general term, seismograph, refers to all types of seismic instruments that write a permanent, continuous record of earth motion. The basic components of a seismograph include a frame anchored to the ground, one or more transducers, timing devices, and a recorder. As the frame moves with the ground, the transducers respond according to the principles of dynamic equilibrium. Signals of horizontal motion in two planes and vertical motion may be sensed either electrically, optically, or mechanically. The motion sensed may be proportional to acceleration, velocity, or ground displacement. The variety of commercially available instruments include the strong-motion accelerograph, peak recording accelerograph, seismoscope and others.

The subject of seismic monitoring of dams, types of instruments used and the analysis of the seismic events recordings is very wide subject and it cannot be covered by a single paragraph, so it may be considered outside of the scope here, and may be treated separately in another work [28] and [29].



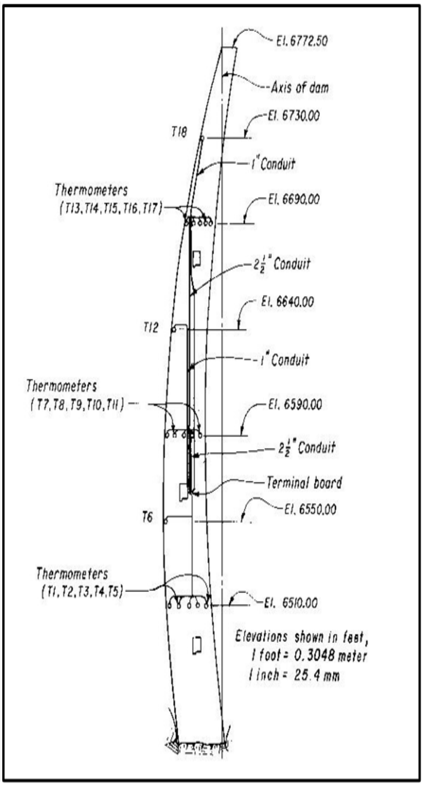

El. 6772.50
Axis of dam
El. 6730.00
T18
1" Conduit
Thermometers (T13, T14, T15, T16, T17)
El. 6690.00
2½" Conduit
El. 6640.00
T12
1" Conduit
Thermometers (T7, T8, T9, T10, T11)
El. 6590.00
2½" Conduit
Terminal board
T6
El. 6550.00
Thermometers (T1, T2, T3, T4, T5)
El. 6510.00
Elevations shown in feet, 1 foot = 0.3048 meter 1 inch = 25.4 mm

Figure 13: (Left) Typical thermometers installation on a gravity dam [27]
Figure 14: (Right) Thermometer installation in an arch Dam [27].

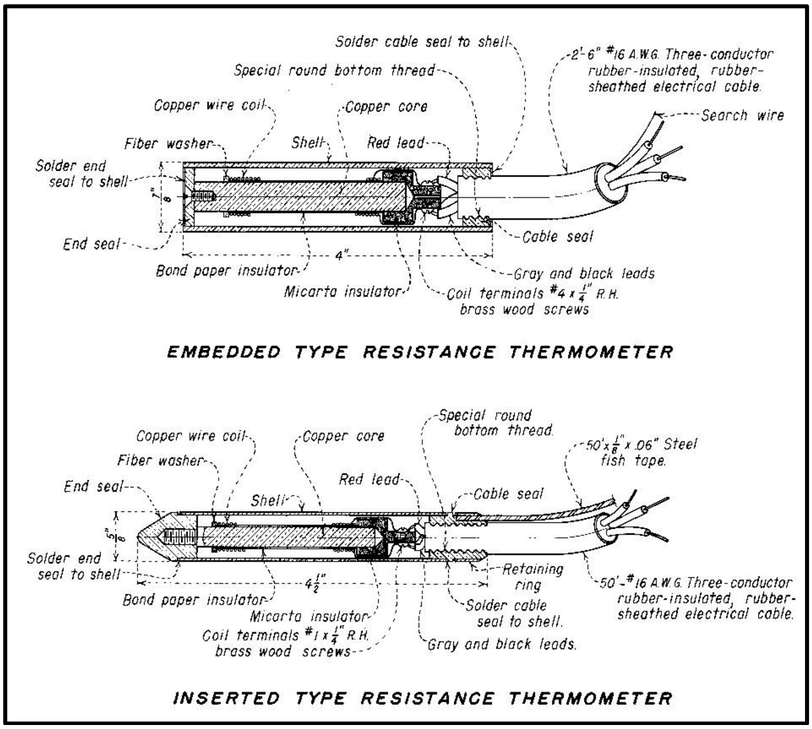

Solder cable seal to shell
Special round bottom thread
2'-6" #16 A.W.G. Three-conductor rubber-insulated, rubber-sheathed electrical cable.
Copper wire coil
Copper core
Search wire
Fiber washer
Shell
Red lead
Solder end seal to shell
7/8"
End seal
4"
Cable seal
Bond paper insulator
Gray and black leads
Micarta insulator
Coil terminals #4 x 1/4" R.H. brass wood screws
EMBEDDED TYPE RESISTANCE THERMOMETER
Special round bottom thread.
Copper wire coil
Copper core
50' x 1/8" x .06" Steel fish tape.
Fiber washer
Red lead
End seal
Shell
Cable seal
5/8"
Solder end seal to shell
4 1/2"
Retaining ring
50'-#16 A.W.G. Three-conductor rubber-insulated, rubber-sheathed electrical cable.
Bond paper insulator
Solder cable seal to shell.
Micarta insulator
Coil terminals #1 x 1/4" R.H. brass wood screws
Gray and black leads.
INSERTED TYPE RESISTANCE THERMOMETER

Figure 15: Details of resistant thermometer [27]

6. Examples of Design and performance of instrumentation systems in existing dams

In order to show the importance of using instruments in monitoring of dam safety, the following examples are described from existing dams.

6.1. Tala dam, an example of concrete dams instrumentation

Tala Hydroelectric Project is a run of the river scheme, located on river Wangchu in South West Bhutan in Eastern Himalayas. It consists of high concrete gravity dam with peaking live storage capacity of 3.2 million cubic meters and an underground power station with total installed capacity of 1020 MW, and complete with all other ancillary structures.

The dam is 92 m high from the deepest foundation with a crest length of 129.50 m and maximum base width of 114.25 m. It has seven concrete blocks with five sluice spillway, one overflow and non-over flow blocks. The sluice spillway is designed to pass standard project flood of 8,575 cumecs and PMF of about10,600 cumecs.

Start of construction was on 1st October 1997 and the first unit was commissioned on 31 July, 2006. The project was executed by the Tala Hydroelectric Project Authority (THPA), a joint venture of the Government of India and the Royal Government of Bhutan. Upstream and downstream views of the completed dam are shown in Figure (16) [30].

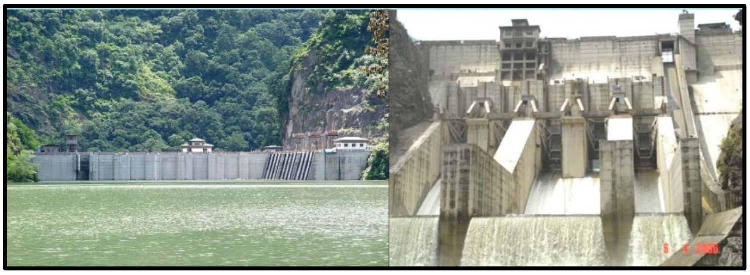


Figure 16: Upstream (Left) and downstream (Right) views of the completed dams [30]

The dam was highly instrumented, and the purposes of this instrumentation and monitoring were in general to maintain and improve safety by providing information to evaluate whether the structure was performing as expected and warn of any changes that could endanger its safety. The provided data were to accomplish the following:

i) Observe temperature development and effects in mass concrete,
ii) Evaluate behavior during construction, filling and operation of the structure,
iii) Verify design and analysis assumptions,
iv) Characterize site conditions before construction,
v) Observe performance of known geological and structural anomalies,
vi) Evaluate performance with respect to potential site-specific failure modes.
vii) Evaluate the dynamic response of the structure due to seismic loads.

Most of the instruments were embedded in blocks 2 and 5 which represent non-overflow and maximum over-flow sections of the dam, respectively, Figures (17,18,19 20).

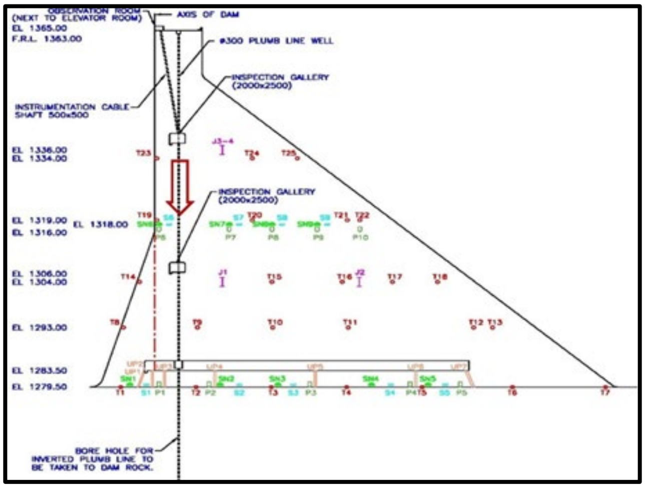
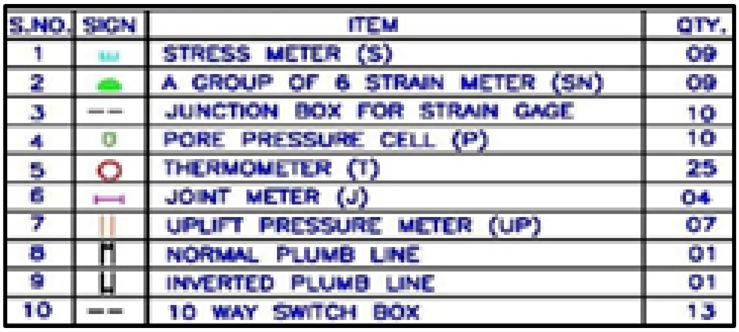


Figure 17: Location of instruments in block 2 [30]



S.NO.	SIGN	ITEM	QTY.
1	ш	STRESS METER (S)	09
2	▬	A GROUP OF 6 STRAIN METER (SN)	09
3	--	JUNCTION BOX FOR STRAIN GAGE	10
4	▯	PORE PRESSURE CELL (P)	10
5	○	THERMOMETER (T)	25
6	⊢⊣	JOINT METER (J)	04
7	‖	UPLIFT PRESSURE METER (UP)	07
8	Π	NORMAL PLUMB LINE	01
9	⊔	INVERTED PLUMB LINE	01
10	--	10 WAY SWITCH BOX	13

Figure 18: Nomenclature for Figure (17) [30]

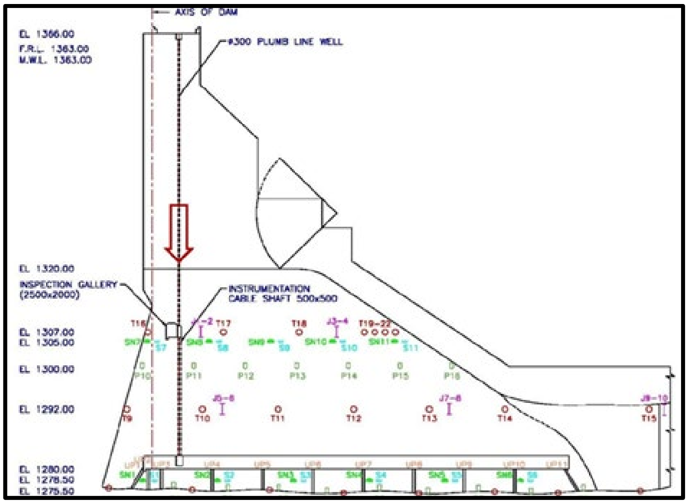
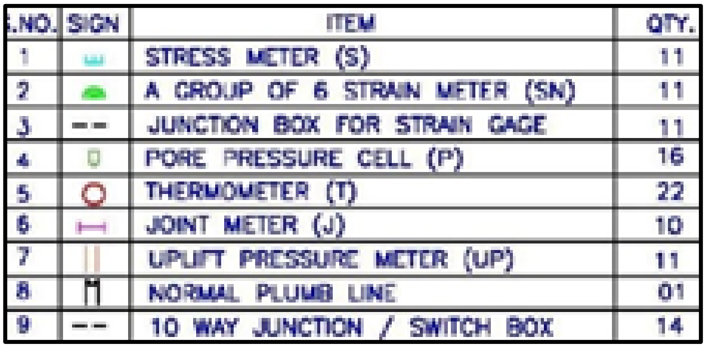


Figure 19: Location of Instruments in block 5 [30]



.NO.	SIGN	ITEM	QTY.
1	ш	STRESS METER (S)	11
2	▬	A GROUP OF 6 STRAIN METER (SN)	11
3	--	JUNCTION BOX FOR STRAIN GAGE	11
4	▯	PORE PRESSURE CELL (P)	16
5	○	THERMOMETER (T)	22
6	⊢⊣	JOINT METER (J)	10
7	‖	UPLIFT PRESSURE METER (UP)	11
8	Π	NORMAL PLUMB LINE	01
9	--	10 WAY JUNCTION / SWITCH BOX	14

Figure 20: Nomenclature for Figure (19) [30]

Joint/ crack meters were installed at construction joints. Multi point borehole extensometers were installed at right and left abutments. Four numbers of strong motion accelerometers have been installed at different locations. The details of instruments being monitored during operation stage is shown in Table (4) [31].

Table 4: Types and Numbers of Installed Instruments in Tala Dam [31]

	Instruments	Proposed	Installed	Working
1	Temperature meter	51	51	48
2	Vibrating Wire pore pressure meter	37	31	30
3	Vibrating Wire stress meter	21	21	20
4	Vibrating Wire strain meter	21 set 126	21set 126	20 set 116
5	Uplift pressure pipe	20	18	13
6	Vibrating Wire Point Boreholes Extensometers	9	8	5
7	Vibrating Wire Crack/ Joint meter	23	19	13
8	Normal Plumb Line(NPL)	2	2	2
9	Inverted Plumb Line(IPL)	1	1	1
10	Strong Motion Accelerometer	4	4	4
11	Vibrating Wire Water level indicator	2	2	2
	Totals	296	283	254
	Total Non-Functional			29

The failed number of instruments represented 10% of the total number of installed instruments. Highest number of failures was in the vibrating wire crack/ joint meters (6 failed out of 19) representing 47% of all such instruments, followed by the vibrating wire point boreholes extensometers (3 failed out of 8) representing 38 %. These high rate of failures may be due to errors in embedding and placing the instruments or due to damaged cables during construction work.

Total of 747 more instruments were installed to monitor the underground structures which were; the desilting chambers, head race tunnel, machine hall, transformer hall, bus ducts and passages, and the pressure shaft manifolds.

These instruments were vibrating wire load cells on rock bolts, vibrating wire load cells on ribs, vibrating wire Piezometers, vibrating wire instrumented bolts, multi-point borehole extensometer of the magnetic anchor type, multi-point borehole extensometer of the mechanical groutable type, total station targets (sections), strain meter (mechanical), strain meter (electronics), and strain gauges.

The continuation of monitoring and analysis of the existing instruments during the operation has been carried out by an independent unit for “Quality Control” who reports to the “Management” directly. As a “Management” policy the instrumentation data and its analysis from safety point of view are reviewed every month in monthly “Progress Review Meetings”. As per the agreement signed with the National Institute of Rock Mechanics (NIRM), India, the monthly data is submitted to them for their in-depth study. The experts of NIRM also visit the project site every quarter and submit the interim as well as Annual Reports.

6.2. Masjed- e- Suliman Dam, an example of an earthfill dam instrumentation

Masjed-e-Soleiman is a clay core rockfill dam on the Karun River in southwestern Iran. The Karun River rises in the Zagros Mountains in western Iran and flows southward to the Persian Gulf. Masjed-e-Soleiman Dam was the second dam on the Karun River which was completed in 2002. Another dam namely Shahid Abbaspour Dam had already been constructed in 1976. Other dams were built after 2002, which were Karun III (2005), Karun IV (2011), Upper Gotvand Dam (2011), location of the dam is illustrated in Figure (21) [32]. Iran Water and Power Resources Development Company has invited domestic and foreign companies to finance, build and operate Karun II Dam and Power Plant project back in 2017, and it can be assumed that this project is.

Karun II Dam and power station are still under construction [33]. The cascade of dams is shown in Figure (22) [35].

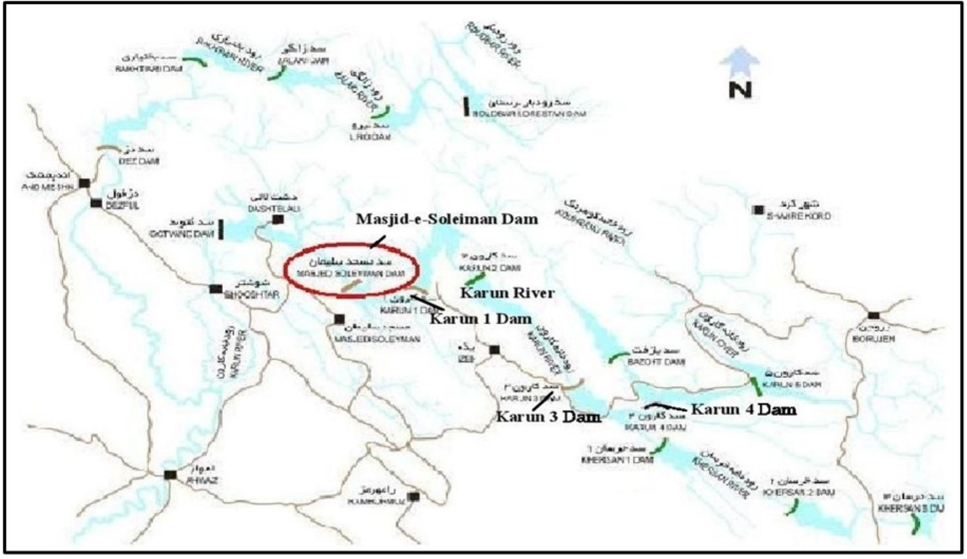


Figure 21: Masjed-e-Soleiman Dam location [34]

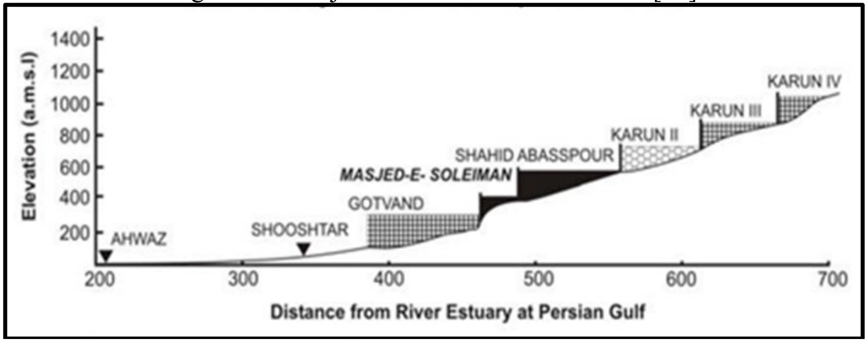


Figure 22: Karun River Cascade Development Schemes [35]

Masjed-e-Soleiman project is a high head plant with a rated head of 145 m and a total capacity of 2,000 MW. The rockfill dam is sealed by a central clay core. The powerhouse is in an underground cavern 260 m long, 30 m wide and 50 m high. View of Masjed-e-Soleiman Dam is shown in the photograph of Figure (23) [36].

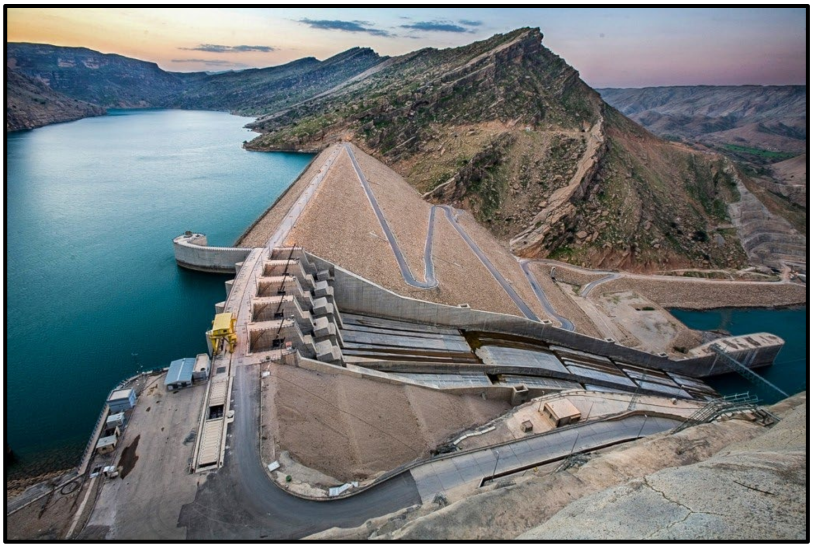


Figure 23: A view of Masjed-e-Soleiman Dam showing the rock fill dam and spillway [36]

The catchment area of Masjed-e-Soleiman reservoir is 27,550 km2. The reservoir has a total storage of 285 million m3 and a live storage of 90 million m3; between elevations 363 (m a.s.l.) and 380 (m a.sl.). Peak flood inflows for the 1,000 year and 100 year floods are 9,300 m3/s and 6,800 m3/s, respectively. The purpose of the dam is to provide river regulation and storage for hydropower generation. Clay core rockfill construction was favored because of the relative high seismicity of the area and the availability of suitable clay and rockfill materials nearby.

The dam comprises a rockfill embankment with clay core and upstream and downstream filters. The upstream slope of the dam is at 1V to 2H and incorporates a rockfill cofferdam with upstream clay membrane at its toe. The overall downstream slope is at 1V to 1.8H and incorporates an access roadway for construction. The clay core of the dam has a minimum width at the crest of 10m increasing in width by 0.4 m for every 1m below crest elevation. Each of the filters has a nominal width of 5 m.

Because of the importance of the dam and its size, it was heavily instrumented. The objectives of the instrumentation and monitoring systems were to confirm the design assumptions and predictions of performance at the construction phase, monitor performance of the embankment during the impounding of the reservoir, confirm safe operation through the life of the dam including the provision of early warning of the development of unsafe trends in behavior and to verify the safe aging of the structure. Typical section in the dam is shown in Figure (24a) illustrating the instrumentation arrangement, and nomenclature is given in Figure (24b) [35].

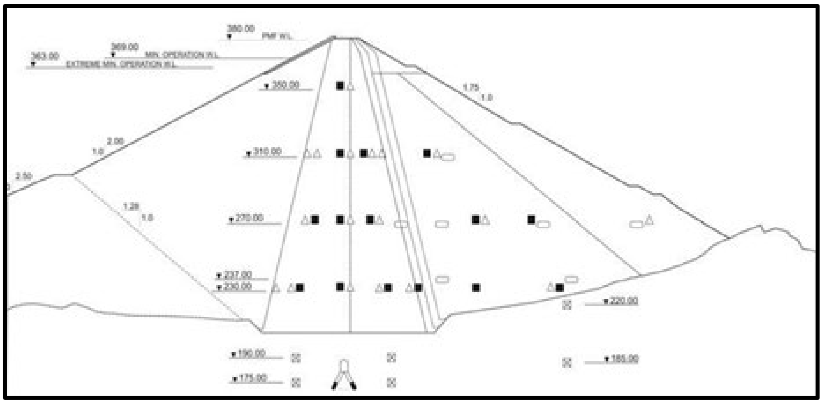
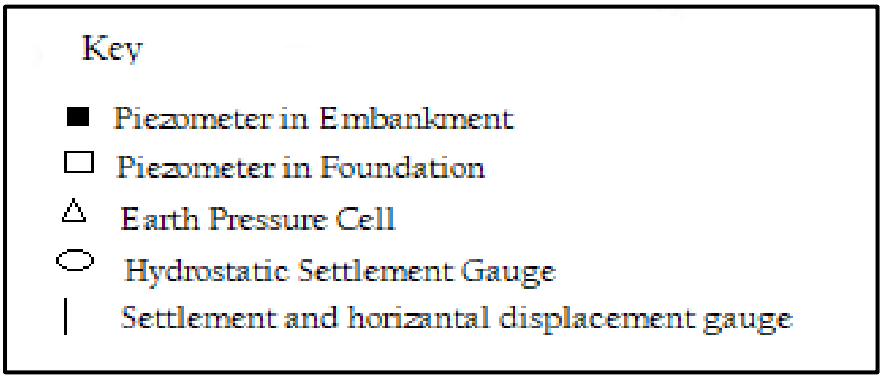


Figure 24a: Typical Instrumentation Arrangement at Masjed-e-Soleiman Dam [35]

Figure 24b: Key to instrumentation arrangement at Masjed-e-Soleiman Dam [35]


Key

■ Piezometer in Embankment
□ Piezometer in Foundation
△ Earth Pressure Cell
⬭ Hydrostatic Settlement Gauge
| Settlement and horizantal displacement gauge

During construction of the embankment, significant number of the instruments were damaged or became inoperable. The reasons for the malfunctions included incorrect installation, damage by construction plant, use of incorrect equipment and faulty instruments. Table (5) sets out details of the instrumentation installed and their operational condition at impounding.

An analysis was carried out of the records of the foundation and core piezometers and earth pressure cells, but the reasons for their failure could not be established indicated from this analysis. Given that the instruments which had failed most were in the upstream shoulder and in the core, it was considered likely that the reason for loss of readings was due to damage of the connections from the instruments to the monitoring points on the downstream face. The cables and tubing had been laid in sand filled trench across the filterand at the transitions between core and filters and shoulder, moreover the cable had been ‘snaked’ in the trench to allow for drawing out due to differential settlement. It was concluded that the friction on the cabling or tubing as the fill continued was too great to allow any movement. This in turn led to high stresses in the cable or tubing and failure at the interfaces where differential settlement occurred.

Table 5: Operational Instrumentation at the end of construction of Masjed-e-Soleiman Dam

Type	Location	Installed No.	Operational No.	Defective %
Vibrating wire piezometers	Foundation Core	14	6	57%
	Foundation D/S fill	5	4	20%
Vibrating wire piezometers	Dam Core	25	18	28%
Pore pressure hydraulic type	Dam D/S fill	10	Unable to monitor	
Standpipe piezometers	Abutments/galleries	22	22	
Casagrande piezometer	DS toe	3	3	
Groundwater observation holes	Abutments	15	15	
Earth pressure gauges	Core	39	30	23%
	DS/ fill	9	8	11%
Hydrostatic settlement gauge	DS/ fill	13	Unable to monitor	
Settlement inclinometers	Core	4 (519 m)	4 (340 m)	35%
	DS/fill	4 (288 m)	4 (263 m)	9%
Earthquake accelerometers	Core crest	1/1	Installed Later	
	Gallery face	1/2		

The reason for the failure of the settlement inclinometers was easier to determine when photographic records were studied and a failed section uncovered. The inclinometer tubes had been joined by use of an outer sleeve as per manufacturer's instructions, but the installer had failed to leave a gap between the tube sections to allow for telescoping of the tubes under consolidation of the fill. Subsequent settlement caused buckling of the tubes at the joints, which meant that it was not possible to pass the probe down the tube.

It was concluded that whilst the instrumentation at Masjed-e-Soleiman Dam was consistent with current international practice, the number of instrument failures were significantly higher than would be normally expected. In particular, the failure of the vibrating wire piezometers, earth pressure gauges and settlement inclinometers, make thorough analysis of the embankment difficult. This demonstrated the importance of adopting good installation procedures.

Analyzing the instrumentation records analysis just before impounding the reservoir showed that the pore pressures within the core were just within acceptable limits, but were dissipating far slower than had originally been envisaged. Because of the possible risk of arching and hydro-fracture of the clay core, it was concluded that the impounding of the reservoir should only be made under strictly controlled conditions to prevent excess hydrostatic pressures in the upstream shoulder. The impounding was staged to allow dissipation of pore pressures in the core to ensure that at no stage would the hydrostatic pressure in the upstream shoulder exceeds the minimum effective stress in the core.

The instrumentation provided sufficient data to determine the behavior in terms of pore pressure and soil pressure, but there was no effective measurement of settlement/ consolidation of the fill due to the failure of the settlement inclinometers and the inability to use the hydrostatic settlement cells until after construction was complete. This was because the hydrostatic instruments required that the instrument houses would be constructed at or close to the elevation of the instruments [35].

One study was performed and published in September 2014 on the total stress and pore pressure development inside the core of Masjed-E-Soleiman Dam to evaluate the quality of measurements of the remaining functioning instruments using the GEOSTUDIO (2007) Finite Element Modeling (FEM) package. The instrumentation records of the dam were used and compared with the numerical simulation and analysis runs. The study consisted of an evaluation of the effective stress as well as settlement/ consolidation analysis up until the end of construction. This study mainly focused on the behavior of the dam within the maximum cross section at chainage 00+260 at elevations 230,270 (m a.s.l.) and 310 (m a.s.l.) during construction.

In order to verify the result of the analysis, the site measurements of Masjed-e Soleiman Dam instruments were used and compared with the results of the Geostudio (2007) software. The analyses in terms of effective stress were combined with consolidation analysis. The numerical analysis results showed good agreement with the instruments data [37].

Another study was done and published in 2013 to show the behavior of the dam from the recorded data. Piezometers that were originally installed in the dam were located in four sections at chainages 00+360, 00+260, 00+160 and 00+430, but the study was restricted to the section at chainage 00+260 only. Registered data in all of the piezometers were up to 5th of September 2009.The purpose was to investigate the possibility of hydraulic fracturing and cracking of the dam at this section.

In section 00+260, the analysis covered the records of all the functioning instruments which were located at elevations 230 (m a.s.l.), 270 (m a.s.l.), and 310 (m a.s.l.).

Generally, the results showed good behavior of the dam and the calculated arching ratios were above the acceptable value of 0.6 and stress condition appeared to be acceptable. Exception to this was indicated by earth pressure instrument No. EP-2202 when compared to pore water pressure piezometer PPE 22, both located at the core at level 270 (m a.s.l.). Registered data of these instruments showed failure conditions at the core and indicated a plastic region incidence; where pore water pressure was in excess of the total earth pressure recorded leading to the possibility of cracking by arching effect.

Moreover, at elevation 230 (m a.s.l.) the registered data by instruments EP-2203 and EP-2205 on the core downstream and shell, respectively were close to the theoretical values of total pressure calculated from the in situ soil height. With respect to the lack of registered data from instrument EP-2204 on the downstream core, it was not possible to compare it with the registered data from instrument EP-2205 in the filter at the same level.

As a conclusion, it seemed that most of the plastic region has occurred in section at chainage 00+260 in the downstream core location near the filter zone and there was possibility of the formation of plastic region at the downstream shell of level 230 (m a.s.l.). It seemed also that there was likely the possibility of plastic region formation at the center of the core and downstream of the core in addition to the shell downstream of 280 (m a.s.l.) of this section. With regard to the very low rate of pore water pressure dissipation in the core, especially at the middle section, it should be noted that this issue had led to the reduction of the effective stress at the dam core. Hence the soil resistance was reduced and plastic condition had occurred. This issue could lead to the formation of the inner cracks and hydraulic fracturing at the core.

Dam stability investigation based on the existing data was performed which confirmed the presence of plastic conditions at some part of the dam body. In the lateral section 00+260, such condition seemed most likely in the downstream core, near the filter, at level 280 (m a.s.l.), and in the lateral section 00+430 of the downstream core and near the filter at level 310 (m a.s.l.) [38].

No further studies of the instrumentation records later than 2009 was available to the writer authors to indicate any improved stability conditions of the Masjed-E-Soleiman Dam.

6.3. Mosul Dam, another example of an earthfill dam instrumentation

Mosul Dam scheme is multipurpose dam project. The dam is located on the River Tigris in the northern part of Iraq about 60 km north west of Mosul city. The earthfill embankment is 3.4 km long; and the main dam across the river channel extends from the right abutment to the spillway structure in the left abutment with totaling length of 2214 m, while the remaining part from the spillway structure to the left end forms much low saddle dam. The maximum height of the main dam is 113 m and it the dam impounds 11.11 km3 of water at maximum operation water level of 330.00 (m a.s.l.) which includes dead storage of 2.95 km3 up to elevation 300.00 (m a.s.l.). Additional capacity of 2.03 km3 is provided between elevation 330.00 (m a.s.l.) and elevation 334.35 (m a.s.l.) to route the 1000 years flood of 12000 cubic meters per second.

The probable maximum flood of 27000 cubic meters per second can be handled at elevation 338.000 (m a.s.l.) before the earthen fuse plug at the saddle dam collapses to allow the concrete emergency spillway underneath operating. This dam is essential not only for providing flood protection but also for supplying irrigation water for the Tigris River basin and for generating electricity from its 750

Mw power station and 200 Mw more from its pump storage facility, in addition to another 60 Mw from its re-regulating dam power station 8 kilometers downstream. Construction of the project began on 25th December 1981 and was completed on 24th July 1986 Figure (25).

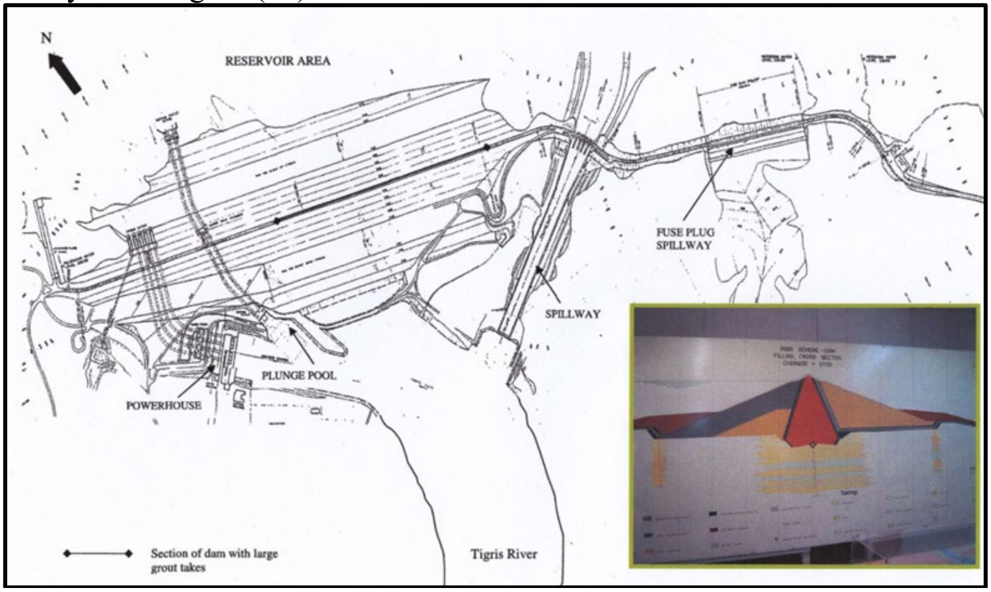


Figure 25: Mosul Dam Layout and Cross section [?]

Mosul Dam cross section is typical of similar embankment dams with outside slopes of 1V to 1: 25H above elevation 330.00 (m a.s.l.) and 1V to 1.75H below this elevation and its maximum base width is 650 m. Compacted clay central core is provided with top width of 7 m and maximum width at its base of 78 m.

The outside shell is of compacted sand gravel mixture protected by 1.0 m thick rock rip-rap at the upstream and oversized gravel layer of 0.6m thickness at the downstream. Very large toe weights were added during the detailed design stage at both U/S and D/S when it was discovered that the stability of the dam was critical during the event of the design earthquake occurrence, Figure (26) [38] and [39] and Figure (27).

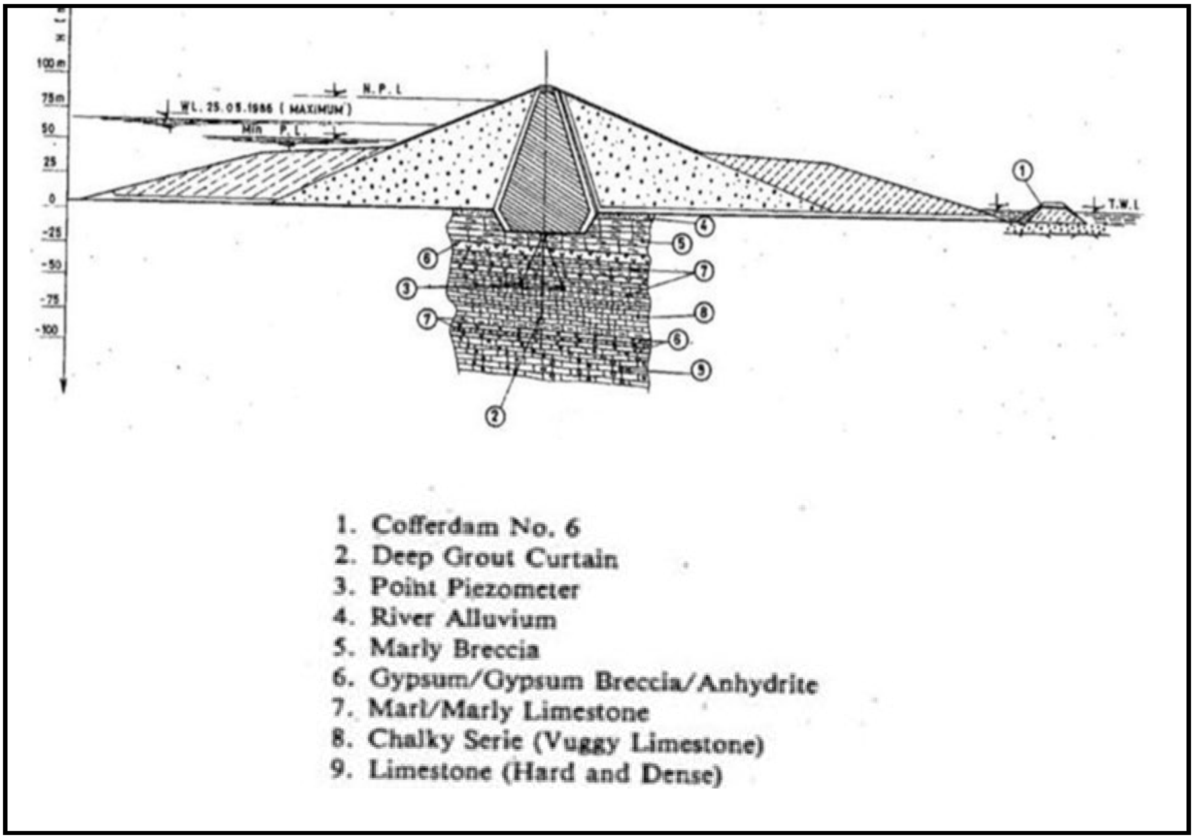


Figure 26: Typical cross section of Mosul Dam [38] and [39]

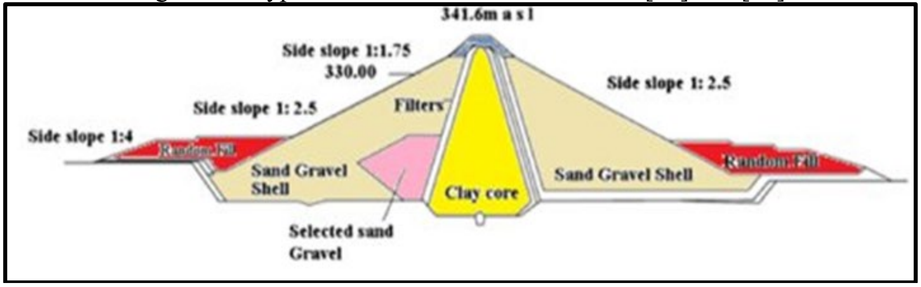


Figure 27. Illustrative cross section of Mosul Dam (from writers the authors collection)

The geology of the dam foundation is very difficult and complex and it is formed of layered sequence of rocks including marls, chalky limestone, and gypsum, anhydrite and limestone. The limestone feature of this geology is the occurrence of karsts and the development of cavities and cracks and fissures.

The foundation anti-seepage works were very important to the stability of the dam to limit the uplift pressure to safe limits and to stop expected dissolution of the karsts rock. These works consisted of deep cutoff trench below the dam core which was filled of the same clay as the core, in addition to blanket grouting up to 25 m thick and deep grout curtain which was thought that it was deep enough to reach the deepest vulnerable brecciated gypsum layer and limits seepage to safe limits; to stop dissolution. The execution of the grout curtain was done from a grouting gallery built at the bottom of the cutoff trench and extended under the main dam from the right abutment till it joined the gallery under the spillway head structure [39] and [40].

A flood wave study was carried out by the dam's designer in 1984 which showed that if the dam failed it would cause a huge flood wave that would flood Mosul city in 1.6 hours and cover it under 24 m of water and reach central Baghdad within 44 hours leaving it under 4 m of water [40,41,42,43].

An updated study on such an event was published in 2016 indicated that the affected population in the Tigris River basin will be more than 6 million people if the water level in the reservoir at failure was 330.00 (m a.s.l.), and more than 2 million if the water level is at elevation 300.00 (m a.s.l.) [43] and [44].

Realizing the great importance attached to Mosul Dam safety and the gravity of its failure the dam was intensively instrumented and great value was attached to the recordings. The design of the instrumentation and monitoring system was developed by the designer during the construction stage with close cooperation and guidance from the International Board of Experts that followed the progress of design and construction.

Various instruments were installed within the dam body, in the dam foundation, and in the downstream area of the dam. Basically, the instruments can be grouped into deformation measurement devices and others to measure pore water pressures. Additionally, deformations of the dam complex can also be measured by geodetic surveying, for which a network was installed.

In the main Mosul Dam, there are seven instrumented cross sections which contain both instruments to monitor deformations and pore water pressures.

Pairs of piezometers are installed also inside the grouting gallery under the dam to monitor the performance efficiency of the grout curtain. Moreover, hundreds of open pipe piezometers were installed in the downstream vicinity of the dam to monitor ground water fluctuations which could result from seepage under the dam. Details of these instruments are as follows:

i) Deformation Measurements

Deformations are measured by electrical extensometers, inclinometers, and geodetic surveys. In each of the instrumented seven sections, extensometers are installed as chains in the direction perpendicular to the dam axis at two different levels in the upper part of the core. There is also an upper chain at elevation 330.00 (m a.s.l.) which represents the maximum operational water level, and a lower chain, close to the base of the core following the excavation topography. Two more chains are similarly installed on both abutments and placed in the direction of dam axis.

In each cross section, there are also three inclinometer/ settlement tubes, in which by inserting a measuring torpedo; settlements along the tube and its inclination can be measured. In each of the seven sections the tubes are installed at 20 m upstream of the axis, in the axis and 35 m downstream of the axis. Those outside of the axis extend into the foundation.

After completion of the dam structure, number of borehole extensometers, type "Extensofor", were installed from the dam crest sub-vertically into to the downstream part of the core. They allow, in a more precise manner than the settlement tubes, to measure deformations within the core. With both types of instruments, the deformation component along the tube is being measured always.

A network of benchmarks was built on the dam surface, both in the cross sections and also between them. From measurement pillars installed outside of the dam structure, by geodetic survey methods, the three dimensional movements of the benchmarks can be determined. For the same purpose, benchmarks are also placed on the ancillary structures, like Spillway, Powerhouse and Bottom Outlets. Additional benchmarks are provided on the dam crest of both the Main and Saddle dams to allow levelling measurements along the crest.

ii) Porewater Pressure Measurements

These pressures are being measured by electrical pore pressure cells and standpipe piezometers. Electrical pore pressure cells are placed in the seven cross sections already indicated. The arrangement in the various zones of the dam is as follows:

Within the core, the cells are at placed in up to four different levels, depending on the height and they are evenly distributed in the upstream downstream direction. These instruments are providing a picture of the pore pressure development within the core. Additional cells are installed in boreholes drilled into the upstream and downstream foundation of the core. Moreover, a limited number of them was placed in the upstream and downstream shell.

Cells are also placed in the downstream blanket drainage layer. Those instruments are at the top of boreholes, which were driven up to 20 m into the downstream foundation and which are containing in general two instruments.

iii) Piezometric Measurements in the Grout Gallery

These measurements are conducted by piezometers which were installed in pairs in the grouting gallery so that each pair at any location measures the upstream and downstream water pressure's difference across the grout curtain indicating its efficiency. In the case when the pressure difference decreases considerably, then such condition marks the development of large seepage path and the deterioration of the grout curtain at that point leading to the formation of cavities in the gypsum-limestone karsts rock. The intension of the designer was to install these pairs at an interval of 72 m, but finally at the insistence of the International Board of Experts the interval was reduced to18 m only, and the total number of these pairs became 76. The readings of these piezometers were recorded using pressure manometers, Figure (27).

These piezometers proved to be priceless during the years from 1985 up to this moment. Late in 1984, it became evident from the readings of these piezometers, that some points in the foundation within the brecciated gypsum layers could not be grouted by any available and possible mean due to the nature and composition of this breccia [39]. Openings in the grout curtain were left during first filling of the reservoir and the following increasing hydraulic head made the dissolution problem in the foundation very acute and threatened the stability of the dam. From 1985 up till now, the curtain and the developing dissolution cavities have been continuously repaired on 24 hr/ 7-day basis. The readings of the piezometers have been the only guide to pin point the critical locations and to grout there. It may be said that the safety of Mosul Dam is wholly dependent on these piezometers and their uninterrupted monitoring.

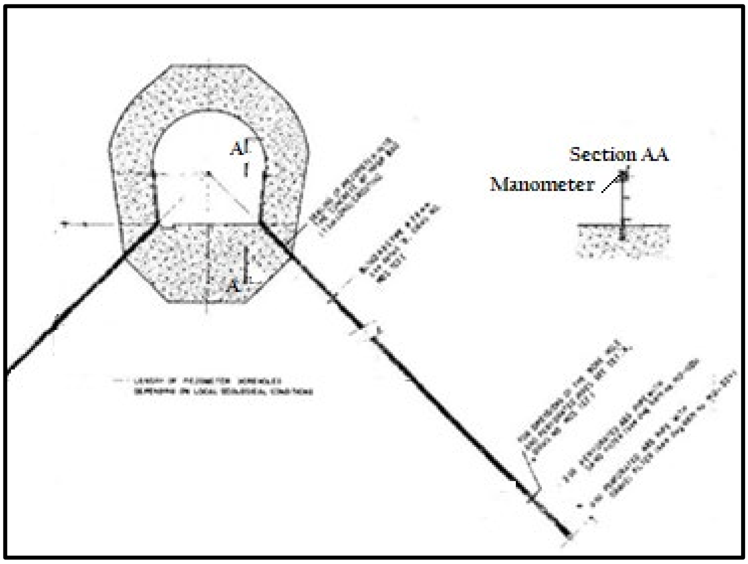


Figure 27: Arrangement in Grouting Gallery [?]

iv) Seepage water measuring devices

When first filling of the reservoir of Mosul Dam was started late in 1984, considerable seepage quantities began to flow from many springs in the left bank downstream of the left abutment of the main dam and from under the saddle dam at the both sides of the spillway chute. Seepage was also expected to occur in the river channel. This condition led to collecting all surface springs into channels and installing weirs for measuring the seepage water quantities. The measurement of transmissivity under the dam in the river section was done by using coffer dam (Point 6, Figure 26) in order to assess the seepage conditions there.

After carrying new grout curtain along the left side of the spillway chute and strengthening the curtain under the saddle dam itself and extending it about one kilometer beyond the dam end, the quantities of seepage were reduced but they do not stop. The need for taking regular measurements of seepage continued and they have been done ever since.

The first weir was installed in point number (1) located at the end of the spillway chute to its right to collect all seepage water crossing under the chute and coming from its left side. The second measuring weir was located at the entrance to the access gallery which leads to the gallery under the dam and it collects seepage water coming from under the left abutment of the Main dam. The third point is point number (3) located at the left side of the spillway and measures the collected seeping water originating from under the saddle dam and emergency spillway. The locations of Point (1) and Point (2) are shown in Figure (28), [45] and [46].

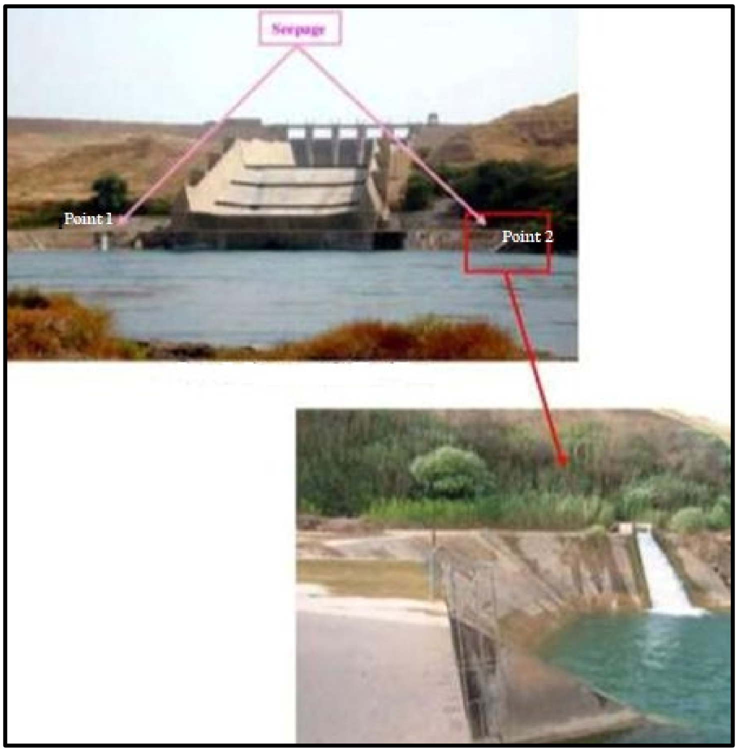


Figure 28: Seepage at left bank (courtesy USACE) [45]

Direct measurement of seepage quantities originating from under the main dam in the river section was not possible by such direct means; as these springs were underwater. So, for adequate surveillance of these quantities during impounding, when water level in the reservoir was rising, it was decided to use the pond created between the toe of the main dam and coffer dam No (6) for this purpose "refer back to Figure (26)".

The crest of this coffer dam was raised above tailwater level created by the controlled flow from the diversion tunnels and its section was strengthened. Set of pumps was operated to measure the quantity of seepage water coming into the pond; while a weir was installed on the crest of the coffer dam to prevent its overtopping when the pumps were not in operation.

It was possible by this way to establish the transmissibility of all the springs under the main dam and correlate this transmissibility with the rising water levels in the reservoir. These controlled monitoring was ended after the flooding of cofferdam No. (6) (Figure 26); when the diversion tunnels were converted to act as bottom outlets [46].

v) Current Instrumentation Situation

The main problem in Mosul Dam is the dissolution of gypsum in the foundation with the possibility of detrimental consequences of settlement of the dam and its cracking. This had caused the initiation of continues grouting program based on monitoring the foundation by the piezometers in the grouting gallery mentioned already "refer back to Figure (27)".

The full program of instrumentation carried out during construction was sizable and included in addition to all the devices; already described large number of piezometers to monitor ground water movements inside the dam and in the downstream to monitor seepage problem. By 2016; however, (40 – 42) % of all the original instruments were out of order. Many of piezometers at various locations inside and outside the grouting gallery were replaced by closed and vibrating wire piezometers. The situation of these piezometers is variable as some stop working due to clogging, but continuous work to flush or replace them is done with efforts to add new ones where required.

Following the rise of ISIS in the middle of 2014 and their occupation of Mosul city and all the area around it, including temporarily the dam site, caused the complete halting of grouting repair works and all observation and recording of the instruments and creating very critical stability conditions. The dam was on the brink of failure during 2015 which led the US Government to help and delegate an interdepartmental team led by USACE to carry out full inspection of the dam and draw up an extensive rehabilitation program. Recommended works, which materialized in concluding a contract in 2016 with Trevi Corporation (Italy) and supervision of USACE included carrying out new emergency grouting drive, full restoration and modernization of the observation instruments, and using the most up to date techniques for measuring, telemetering the acquired data on real time basis, and establishing a comprehensive data base to which all the previously manually recorded historical data were added. This data base included; among other things data of 680 observation wells from previous works. The data base was to be used in supporting the grouting works carried out then and to be continued in the future and to establish the safety conditions of the dam at any time from the newly upgraded instrumentation system.

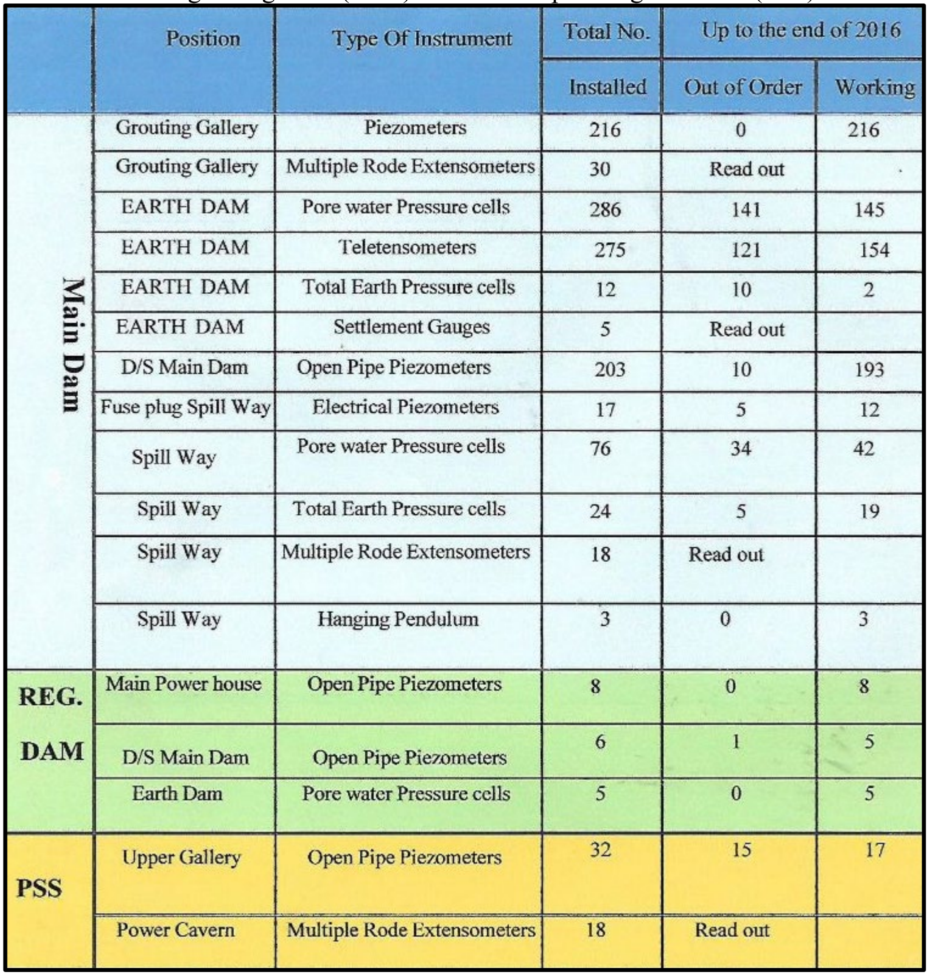
At the present, the actual situation of instrumentation of the main dam, the re-regulating dam and the pump storage scheme is given in Table (6), which indicates types and numbers of the instrument used.

A new team of engineers to staff the safety control section in the management organization of the dam was trained by USACE on the use and follow up the newly acquired modern instruments. This team includes six veterans from the old dam safety section and many newly graduated engineers who became acquainted with all the details and necessary skills.

The grouting of emergency grouting program was completed successfully in 2018 with great support from the refurbished old instruments and the newly installed devices and digital logging, transmitting and recording intelligent systems.

The Trevi contract which included other than, grouting and instrumentation, the rehabilitation of the guard gate of one of the bottom outlets materialized in executing total quantities of new grouting works of drilling 525,000 meters in about 5,000 grouting holes with total grouting material consumption of 41,000 m3 of grout mix weighing 27,000 tons. This great amount of work would not have been possible without the use of down the hole push cameras, new boreholes inclinometers' chains, down the hole flowmeters, and new pressure gauges on top of the piezometers and preventers. In addition to that, the use of real- time telemetering of the data to the control station for logging and presentation facilitated allows taking immediate and fast decisions on required actions.

Table (6): Numbers and Types of the all the instruments functioning now in the Main Dam, Re-regulating Dam (REG) and the Pump-Storage Scheme (PSS)



	Position	Type Of Instrument	Total No.	Up to the end of 2016	
			Installed	Out of Order	Working
Main Dam	Grouting Gallery	Piezometers	216	0	216
	Grouting Gallery	Multiple Rode Extensometers	30	Read out	
	EARTH DAM	Pore water Pressure cells	286	141	145
	EARTH DAM	Teletensometers	275	121	154
	EARTH DAM	Total Earth Pressure cells	12	10	2
	EARTH DAM	Settlement Gauges	5	Read out	
	D/S Main Dam	Open Pipe Piezometers	203	10	193
	Fuse plug Spill Way	Electrical Piezometers	17	5	12
	Spill Way	Pore water Pressure cells	76	34	42
	Spill Way	Total Earth Pressure cells	24	5	19
	Spill Way	Multiple Rode Extensometers	18	Read out	
	Spill Way	Hanging Pendulum	3	0	3
REG. DAM	Main Power house	Open Pipe Piezometers	8	0	8
	D/S Main Dam	Open Pipe Piezometers	6	1	5
	Earth Dam	Pore water Pressure cells	5	0	5
PSS	Upper Gallery	Open Pipe Piezometers	32	15	17
	Power Cavern	Multiple Rode Extensometers	18	Read out	

The final conclusion of the USBR team who engineered the works and supervised its execution is the need of the dam for a deep diaphragm as permanent solution to the seepage problem or to continue grouting till the end of the dam life; may be after 50 years.

The present arrangement in following the continues grouting operation is that all measured data from the observation devices are transmitted to computers every 15 minutes and these are logged and can be checked instantaneously and daily with the grouting sections and dam management and, if any anomaly is observed then immediate response is taken.

Safety of the dam as a whole is followed by the coniferous recording of all other instruments which are transmitted and logged in similar way and a monthly report is compiled for immediate action and future reference.

To give an idea of the cost of the whole rehabilitation program, the U.S Government paid nearly $125 million for the U.S. Army Corps of Engineers to provide engineering work and technical advice. While the Italian firm Trevi S.p.A. was paid nearly $410 million for performing the rehabilitation works. This amount of money was facilitated by a World Bank loan to the Iraqi Government.

"Strips and Strips", the newspaper published by the U.S Army in Germany published an article on the contribution of the USACE to this work. Adamo who is the senior writer of the current paper contributed to this article through an interview with him which expressed appreciation to the high quality of the work done [47].

7. Summary Points and Conclusions

7.1. Instrumentation of dams is a powerful tool in evaluation of dam safety and its good performance. Together with visual inspections and other measurements made at the site, instruments can discover early signs of abnormal behavior. Careful examination of instrumentation data on a continuing basis may reveal a possible critical condition. Conversely, instrumentation may be one mean of assuring that an observed condition is not serious and does not require immediate remedial measures. Therefore, on this basis it may be concluded that the high cost of installing and following up such instruments and analyzing their recordings can be justified. A balance must be drawn, however, between the level of safety assurance instrumentation provides and its cost in case of budgeting limitation.

7.2. The scope of available monitoring devices available today includes instruments for measuring and recording of such variables such as: deformation, seepage/ leakage quantity and quality, uplift pressure, pore water pressure, temperature, reservoir level and seismicity. The choice of kinds and numbers of instruments for a planned or existing dam depends mostly on the type of the dam, whether embankment or concrete dam, and the importance of the dam and its risk category classification.

7.3. In any satisfactory instrumentation program, the number and types of devices should be adequate to monitor structural and functional performance, and the devices should be installed in dams where complex or unusual site conditions have been noted. Furthermore, the instruments installed in a dam must be observed and their recording onsidered. From time to time, they should be examined by specialists to determine if they are sufficient to help evaluating the performance of the dam. Additional instrumentation may be installed if needed to confirm suspicious trends or to explore an indicated potential adverse trend. Moreover, observation of monitoring devices, at predetermined frequency, should be done by trained personnel who can recognize signs of problems and report them immediately to the higher technical level for further action. This frequency can be increased or decreased according to the development of conditions at site. A conclusion can be drawn here that a fruitful instrumentation program depends greatly on the human factor, i.e. having at site personnel of the proper expertise. It is also recognized that the next step in such program depends also on specialists of high experience working at the higher management level who undertake the analysis of obtained site data and can infer any conditions that may require upgrading works to the dam to enhance its safety conditions.

7.4. Collection of data from installed instruments in dams have progressed over time. Manual reading of all instruments and recording the results for conversion later on into tables and graphs for analysis in the office has been the conventional method. If the number of instruments is very large, and if very frequent readings are required, then the job of reading and recording becomes difficult and cumbersome. Collection of data from many types of instruments can be done nowadays by providing them with electronic transducers and they are read by digital stand-alone loggers so that all measurements are downloaded into computers in the office for later analysis. Real time networks which allow the automatic retrieval of measurement from remote locations or remote sites on continuous basis are also available. In both cases, it is desirable and highly recommendable to do frequent inspections of stand-alone loggers and the automated real time systems. This is necessary and vital to insure their good performance, and such inspection visits will eliminate the possibility that the instruments are malfunctioning and the received data being not correct. Incorrect received data can give false impressions and may lead to wrong operational procedures which endanger the dam safety instead of improving it.

7.5. There exists today a great variety of instrumentation devices for the measurements of variables in dams. Following so many cases, it is noticed that certain percentage of the instruments fail either as result of construction activities at sites, improper installation or even selecting cheap instruments from unknown manufacturers. Any device, which fails is considered as final loss and its replacement by new one can cost additional money. It is therefore highly recommended to select renown manufacturers to supply these devices, and that the installation at site is done under close supervision by the manufacturer’s personnel, and to include in his contracts an item for sharing the cost of replacement; if it is proven that failure was attributed to him.

7.6. Accumulated recorded data can help in evaluating the behavior of dams and if they are free from possible dangers. Examples have been given in the present work for some of such dams. The Tala Concrete Dam case shows that the rigorous follow up of dam safety conditions reveal good performance of the dam; even with the fact that the percentage of failed instruments was not small. In the Masjid-e- Suliman Embankment Dam example, the number of defective instruments was high and the analysis of the reading of the remaining instruments after few years showed unsatisfactory behavior in certain locations of the dam body. This leads us to question whether different results could have been obtained; had all the instruments performed satisfactorily. Finally, in the Mosul Dam case, it is considered that the instrumentation was done in the best possible manners from point of view of the design to the selection of instruments types and their installation. They all showed relatively good performance of the dam. It was also concluded that without the piezometers in the grouting gallery it would have been impossible to measure the deteriorating efficiency of the grout curtain and to follow the formation of dangerous solution cavities in the foundations. Detection of such cavities by piezometric measurements was essential in the remedial grouting operations over almost four decades which have restored the stability of the dam. The recent upgrading of the dam instruments, which was carried out in conjunction with the grout curtain repair work, underlines the high value of modern instrumentation devices and observation systems; recently installed for the future safety of the dam.

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Dam Safety: Use of Seismic Monitoring Instrumentation in Dams

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Abstract

Seismic instrumentation of dams and reservoirs sites is accepted today as a valuable tool to understand significant seismic hazards facing existing dams or future planed dams. With the advent of digital seismic accelerometers and recorders, it can now be used today as an integral part of dam safety monitoring systems. Outputs of these instruments help in understanding the dynamic response of dams during earthquake, assessing the damage caused by such events and determining required upgrading works necessary for existing dams and designing of safer dams in the future. Measuring and recording by strong motion seismographs covers the induced Peak Ground Acceleration (PGA), velocity and displacement recorded on time scale to indicate the intensity and frequency of ground vibration at the site during seismic events. Seismometers for such measurements and recordings have undergone considerable evolution and there exist today a variety of these instruments with high degree of refinement which can even provide for remote sensing. In this work, this development is outlined and examples of seismic instrumentation in strategic dams are described. Damages to actual concrete and embankment dams of various types are described indicating the associated PGAs experienced during the mentioned earthquakes. Damages in the form of cracking, increased seepage, additional settlements and displacements are described to show type and extent of possible consequences of such events on dams. The reached conclusion is that seismic instrumentation systems are desirable and highly recommendable for all types of dams; existing and future ones and their high cost is justified by the service they provide.

Keywords: Seismic instrumentation, seismic hazard, accelerometers, safety

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monitoring systems, dynamic response, strong motion seismograph, ground vibration, Seismometers, remote sensing.

General

History of recorded dam failures and incidents due to earthquake ground shaking indicates that many dams had failed or badly damaged by such events during the second half of the nineteenth century and first half of the twentieth century [1].

Design criteria and methods of dynamic analysis of large concrete and embankment dams have undergone substantial changes since the 1930s after Westergaard had developed his theory of structures based on the elastic theory, and loading due to earthquake action was introduced into their design. Until the 1960's, seismic analysis of dams consisted essentially of the use of the seismic coefficient method, in which a static horizontal inertia force was applied to the potential sliding mass in embankment dams, or center of gravity of concrete dams, in an otherwise conventional static limit analysis. The magnitude of the inertia force was chosen on the basis of judgment and tradition and it was represented by a seismic coefficient multiplied by fraction of the dam weight. Typically, a seismic coefficient value of 0.1g was used for most dams. In exceptional cases in Japan and Iran, slightly higher values were considered. The seismic coefficients had no clear physical relation with the design ground motions and the seismic hazard at the dam site. Moreover, the dynamic response was determined by a pseudostatic analysis, which does not account for the dynamic characteristics of the dam.

Due to the simplicity of this method it remained in use, although it had no scientific basis, until late 1980s, but gradually has been replaced, especially for moderate or significate risk dams, by more rational use of analysis of the actual response of dams. This has raised the need for advanced monitoring instrumentation capable of recording peak ground acceleration and its variation with time during the event,

Early seismic instruments for dams were developed during the 1930s by the United State Bureau of Reclamation (USBR). Most technological advances in this field, however, have occurred during the 1970s and after; when number of vibration measuring devices have been developed and of these, the seismograph (accelerograph) is now the most commonly used seismic instrument. This instrument consists of a sensor (seismometer or accelerometer) and a mechanism for producing a permanent record of the vibration applied to the sensor. Nearly all seismic instruments in use today utilize servo- accelerometers that have the ability to measure motion in a single horizontal, vertical, or transverse plane. Most of the devices are considered “strong motion” instruments that record significant movement, as opposed to micro- seismic activity that requires the use of signal conditioner or enhancers to magnify the motion to recordable level. The first strong motion instrument installed in USBR was Hoover Dam in 1936 [2].

Obtaining data on ground motion during earthquakes to which all types of structures such as high-rise building, power stations, bridges and dams is very important for the investigation of the dynamic behavior of these structures and for define designing criteria of future similar structures. This has given rise to the development of networks of strong motion instruments installed in various countries of the world and especially in seismic regions to gather such information.

In the beginning of the 70s, networks of strong earthquakes recording instruments were installed in several seismic regions in the world such as USA, Japan, Italy, former Yugoslavia and others. This example was later followed by several other countries like Mexico, New Zealand, Iran, Turkey, Greece and others, thus at present, there is a relatively high number of such networks. Today, earth is being constantly monitored by over 20,000 strong motion seismometers deployed throughout the world, most of which transmit data in real time. As early as the late 19th century, Rebeur Paschwitz had understood the fundamental advantage of being able to study an earthquake from several points on the globe, and recent technological innovations have made it possible to record, digitize and relay seismic data to remote observatories.

Even with the large number of strong motion seismometer aforementioned, they are still not sufficient to cover all the seismically active regions in the world and to provide sufficient quantity of usable data. Therefore, large number of countries in the engineering practice apply records obtained by other countries. But, having in mind that earthquakes are characterized by:

the frequency and amplitude content, which depend on the geological and tectonic structure of the seism region,

the magnitude, and/or the intensity of the earthquake,

the origin depth,

the epicentral distance. Then it is obvious that they differ from those recorded in other areas, even in cases when earthquakes of the same intensity are considered. Therefore, it is necessary to use records from the actual seismogene region, or if used from another region, then one should be careful, and, if possible, use records from a region having similar seismo-tectonic characteristics [3].

This development, however, has not precluded the need for installing strong motion instruments at medium and significant risk dams' sites to define the strong motion parameters at such sites which are defined and modified by geology and topographic conditions at their sites [4].

The outdated concept that seismic instrumentation of dams and reservoirs' sites is only a research tool has given way to the modern concept that seismic instrumentation is necessary to understand significant hazard dams' behavior in seismic areas. It is also desirable in traditionally nonseismic areas. With the advent with digital seismic equipment, it can now be an integral part of dam safety monitoring works. The digital earthquake data can be gathered by site personnel and remote control centers by use of computer programs. When the digital instrument is installed with modem and communication means, then remote access from several offices is available.

These devices typically consist of three mutually-perpendicular accelerometers, a recording system, and triggering mechanism. To prevent accumulation of unwanted data, the instruments are usually set to be triggered at accelerations generated by nearby small earthquakes or more distant, larger earthquakes. They are expensive, especially considering that multiple instruments are necessary to record dynamic response at several locations on a structure, a foundation, or abutments. The devices must be properly maintained, so that they operate if an earthquake takes place.

Seismic instrumentation's installation should be considered on a case-by-case basis depending on; dam design, foundation materials, and methods of construction. Site-specific seismotectonic data needs should be weighed against potential benefits before any seismic strong motion instrumentation is adopted [5].

2. Seismographs and Their Use in Dams

The general term, seismograph, refers to all types of seismic instruments that record a permanent, continuous record of earth motion. The basic components of a seismograph include a frame anchored to the ground, one or more transducers, and a recorder. As the frame moves with the ground, the transducers respond according to the principles of dynamic equilibrium. Signals of horizontal motion in two planes and vertical motion may be sensed either electrically, optically, or mechanically. The motion sensed may be proportional to acceleration, velocity, or ground displacement. A triggering mechanism is provided to prevent accumulation of unwanted data and the instruments are usually set to be triggered at accelerations generated by nearby small earthquakes or more distant, larger earthquakes. These equipment are expensive, especially considering that multiple instruments are necessary to record dynamic response at several locations on a structure, a foundation, or abutments. The devices must be properly maintained, so that they operate if an earthquake takes place.

An example of seismograph is shown in Figure (1), which is of Kinemetrics seismograph, formerly used by the USBR [6] and [7].

The earliest form of seismometers was known as early as 132 A. D in China using pendulum as the principle seismic measuring device. The history of seismometers development from that time up to 1900 is given in the book, "Early History of Seismometry to 1900"[8].

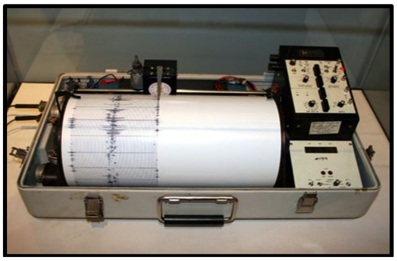

Figure 1: Kinemetrics seismograph, formerly used by the United States Department of the Interior [6] and [7]

Two main types of seismographs exist today as far as the recording of obtained data are concerned namely; analog recording seismographs and the digital recording seismographs.

In dams, a digital seismograph can be part of other data acquisition systems installed in such dam such as a trigger device for piezometer recordings or for slope stability during an earthquake, a telephone calling system to report an earthquake event, or a multi-recorder installation to study the response of particular appurtenant dam structure.

According to USBR practice, strong motion seismographs are categorized according to their placement location.

i) Free field instruments

It is recommended that such instruments are located near both abutments at the toe but at such distance beyond any significant influences of the dam on the recorded ground motion.

ii) Input motion instruments

They are to be placed at the downstream toe and to the abutments as close to the dam as possible. Most of these instruments are placed in prefabricated housing on concrete pads firmly secured to the underlying rock or surface material, but normally finding suitable locations at the toe is difficult due to various tail water conditions, and difficulties in locating them on the abutments can be due to restricted access due to topographic conditions. In abutment areas, an ideal installation in a spatially restricted area would be a small chamber in the natural material where maintenance problems could be minimized. Siting interior or subsurface input motion instruments consist of boreholes instrumentation in the foundation within selected galleries in concrete dams. Drainage and grouting galleries, when excavation in earth dam foundation, can be utilized as input motion sites for strong motion instruments.

iii) Response instruments

These are located on the dam to determine dam response to the vibration. Ideally, one or two response instruments are installed on the crest of both earth and concrete dams. The primary location is where maximum deformation during strong motion is expected, usually at the maximum section. A secondary section may be about one-third of the crest length from an abutment, such a location is basically for backup purposes. If a dynamic analysis of the structure has been done prior to strong motion instrument deployment, the response instrument location may be specified based on the analysis. Specified areas would be where lower safety factors and higher loads are expected. These locations are site specific for each structure. The locations for earth dams depend upon zoning geometry of the dam, types of materials used in the zones, and nature of the foundation; while for concrete dams depend upon type of the dam (gravity or arch), geometric configuration of the dam and nature of foundation materials [9].

One possible arrangement of instruments location is shown in Figure (2). This is given only as an example, but actual numbers and arrangements may be done after careful assessment of the needs based on type and importance of the dam and the seismic region where it is located, [10] and [11].

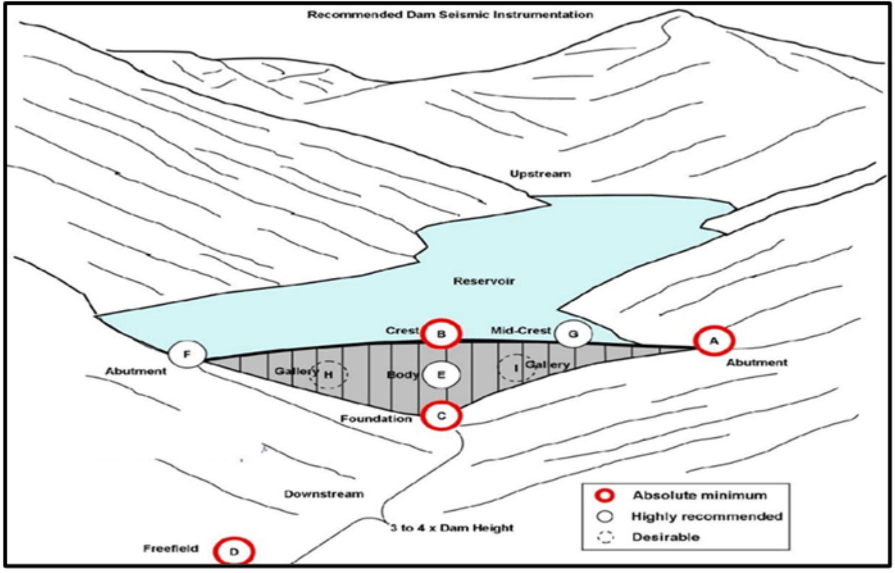

Figure 2: Example of Dam Seismic Instrumentation [10 and 11]

Basically, vibration measuring devices consist of sensors, signal conditioner, and a recorder or storage medium.

i. **Sensor:** The sensing devices used are electromechanical units that respond to motion and produce an electrical signal that is, within limits, proportional to displacement, velocity, or acceleration. Most sensors are models of single degree of freedom, spring-mass dashpot system. The measurement is usually made of the spring extension or compression with the resonant frequency and the damping of the system so proportioned as to produce an electrical signal that is an analog of either acceleration, velocity or displacement. Sensors may be located at the recording unit or placed in a remote location; such as in a drill hole, or elsewhere. Signals are transmitted by a coaxial cable to the signal conditioner. It is common for sensors to contain a starter or triggering device that activates the sensor at some predetermined acceleration, such 0.01 gravity. The sensor would then continue to operate as long as the motion is greater than that value, and for short time thereafter. These sensors act as seismic alarm devices (SAD) and they are installed where a display of peak acceleration is required immediately following an earthquake. The instruments measure acceleration, triaxially, vertical, longitudinal and transverse directions, and they are available in digital and analog models.

ii. **Signal Conditioner** The term “signal conditioner” refers to all units and devices placed between the sensor and the final output data recorder. These devices are usually power amplifiers that are required to change the micropower signal level from the sensor to the macrolevel required to activate the recorder system. Signal conditioners also usually include sensitivity controls to permit a desired level of recorded signal.

Signal conditioning equipment may also include analog integration units for conversion of one measured parameter to another. The equipment may physically be part of the sensor unit, part of the recording unit, or may be separately packaged.

iii. **Recorder** The recording unit is the final device in the system and is located in a protected environment in a secured area. The recorder presents the output in some usable form for evaluation and/or further use. Most recorders now being used produce historical records of the input phenomenon versus time as a paper record. Such records provide a quick method for visual inspection, and permit a rapid evaluation of peak amplitudes and other values. Detailed study of data in this form requires point- by point transcription of values for future computation. Automatic data handling can be obtained for the use of magnetic tape recorder and later playback on to computing systems. The most desirable system includes an output of both direct reading paper records for rapid field inspection; plus a magnetic tape for direct storage, which allows for later computer processing. Most recorders also provide a timing base that is recorded along with the sensor signals as a reference for determining the frequency of the vibration.

Generally, recorders may be provided as analog recording units or digital recording units. The analog type record data on 70 mm film which must be recovered under low light conditions and chemically processed to develop the film. A component diagram of such recorder is shown in Figure (3) [12].

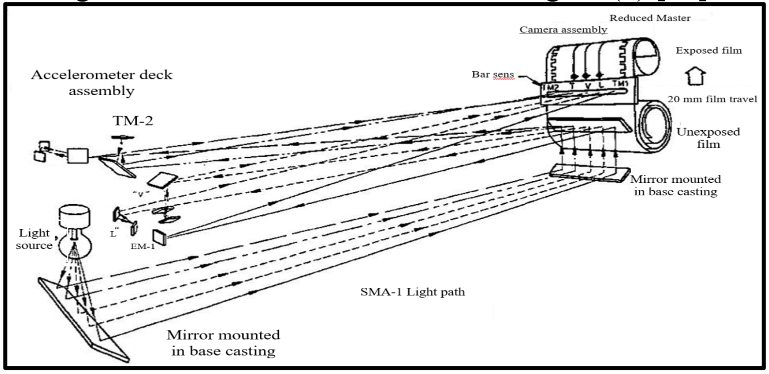

Figure (3): Analog/accelerograph component diagram showing working principles [12]

Digital recorder continuously digitalizes the three internal force balance accelerometers and stores data in solid state memory. The general arrangement of a digital accelerograph box is presented in Figure (4a), and a typical plot of such accelerograph is shown in in Figure (4b) [12].

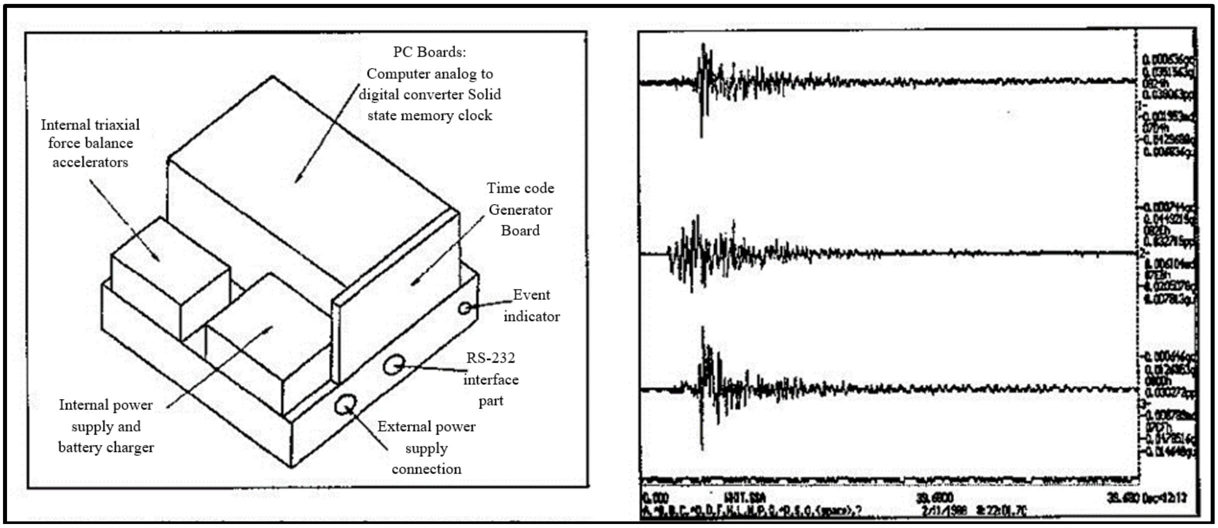
PC Boards:
Computer analog to digital converter Solid state memory clock
Internal triaxial force balance accelerators
Time code Generator Board
Event indicator
RS-232 interface part
Internal power supply and battery charger
External power supply connection

Figure 4: (a) on the left shows the general arrangement of a digital accelerograph, (b) on the right is typical plot of such accelerograph [12]

Development of strong motion instrumentation from the early Chines instrument up to now has gone very far and it is possible now that at each measuring point on a dam or other important structure a high dynamic mechanical force balance accelerometer is deployed. These devices collect raw acceleration data and transfer it to Data Acquisition System (DAS). From the DAS, the data is collected and sampled. It can then be transmitted to an offsite location or stored locally for automatic intelligent processing. The reliable level of remote monitoring eliminates the need for site visits for structures in remote locations. The system allows for tools to remove mundane data, noise, thermal, or other unwanted effects before storage, and make data interpretation easier, faster, and more accurate. Diagnostics convert abstract data signals into useful information about the structural response and conditions [11].

3. Modern Dam Monitoring Systems and Examples

Great progress has been done in the world today in the field of seismic monitoring of dams. The high cost of seismic monitoring systems of large strategic dams utilizing available modern technologies of recording and transmitting data to remote control centers is justified by the great value attached to these dams. Two cases of such progress are presented in the following;

3.1 Old Aswan Dam and High Aswan Dam, Egypt

An example of advanced monitoring systems applied to Old Aswan and High Aswan Dams in Egypt is given here to show the type and level of technological sophistication reached in the field of seismic safety monitoring of dams and to explain the techniques used.

The contract was awarded in 2011 to Ref Tek, a division of Trimble Geospatial for the design, supply and commissioning of a strong motion instrumentation network for both dams. The network was installed with the support of Noor Scientific & Trade Co. and completed in 2013.

The Old Aswan Dam is a 54 m high and 1900 m long gravity buttress dam whose construction began in 1899 and it was completed in1902. The buttress sections accommodate numerous gates, which were opened yearly to pass the flood and its nutrient-rich sediments, but without retaining any yearly storage. The dam was constructed of rubble masonry and faced with red ashlar granite. The design also included a navigation lock of similar construction on the western bank, which allowed shipping to pass upstream as far as the second cataract, whereas portage overland was previously required. When constructed, the Old Aswan Dam was the largest masonry dam in the world; nothing of such scale had ever been attempted. The initial construction was found to be inadequate for development needs, and the height of the dam was raised in two phases; five metres between 1907-1912 and nine metres between 1929-1933. Generation of electricity was added. The Old Aswan Dam supports now two hydroelectric power plants, Aswan I (1960) and Aswan II (1985–1986). Aswan I contains 7 X 40 megawatts generators with Kaplan turbines for a combined capacity of 280 megawatts and is located west of the dam. Aswan II contains 4 x 67.5 megawatts generators for an installed capacity of 270 megawatts (360,000 hp) and is located at the toe of the dam.

When the dam almost overflowed in 1946, it was decided to build a second dam, which is the High Aswan Dam in the upstream rather than raise the dam for the third time, Figure (5). With the construction of the High Aswan Dam upstream, the Old Dam's ability to pass the flood's sediments was lost, as was the serviceability provided by the navigation lock. Aswan High Dam, is a huge rockfill dam, located on the Nile River north of the border between Egypt and Sudan. The Dam, known as Saad el Aa’li in Arabic, was completed in 1970 after ten years of work.

The dam is a massive structure containing 18 times the material used to build the famous Pyramid of Cheops at Giza. It is 3,600 meters long 980 meters wide at the base and 111 meters high above the river level. Figures (6,7) show some views of the dam and the power station.

The two Aswan Dams benefit Egypt by controlling the annual floods on the Nile River thus preventing the damage which used to occur along the flood plain. The High Aswan Dam increased cultivable land by 30% and provides about a half of Egypt’s electrical power. The total installed capacity in its power station is 2100 MW provided by 12x175 MW Francis type turbines.

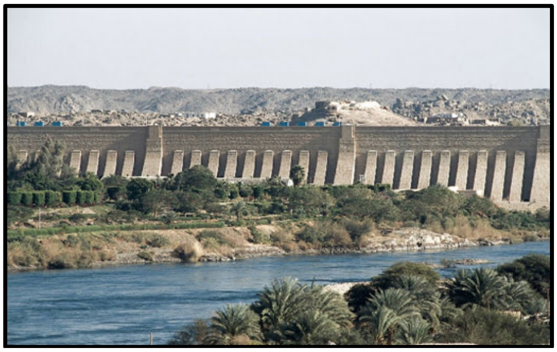

Figure 5: Downstream view of the old (low) Aswan Dam [13]

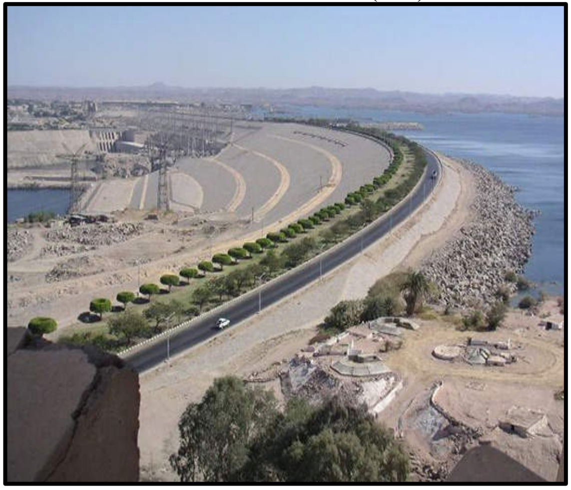

Figure 6: An overview of the High Aswan Dam (water Technologies) [14]

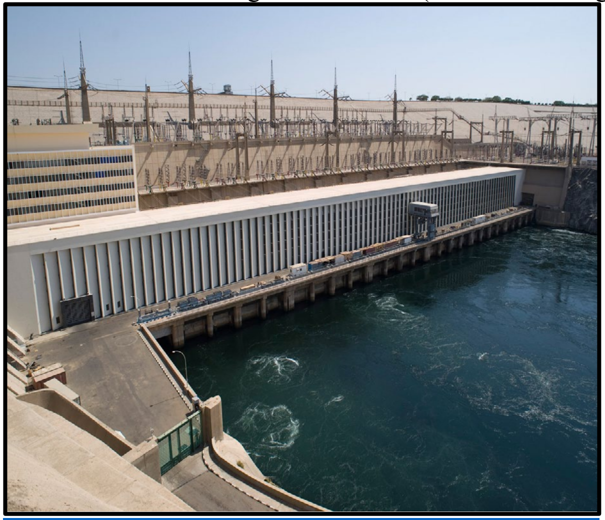

Figure 7: Downstream view of the power station at High Aswan Dam [15]

The a/m information is given to show the extreme importance and strategic nature of these infrastructures; this led the Egyptian National Research Institute of Astronomy and Geophysics (NRIAG) to agree with the Aswan and High Dams

Authority (HADA) that an array of strong motion accelerographs should be installed to record seismic activity in the area, in order to better monitor and study its effect on the structures of the two dams.

NRIAG criteria for the strong motion array consisted using the following:

i) Remote and fixed strong motion accelerograph stations consisting of accelerometers and digital recorders,
ii) Communications network; and software to receive and analyze the seismic data.

Fort the High Aswan Dam, the instrumentation installed were five low noise, high resolution and strong motion recorders (Model 130-SMHR); four in the High Dam structure, and one (free field station) located on bedrock at the north side of the Dam base. The illustration in Figure (8) below shows layout of the array inside the dam. The High Dam has three galleries (tunnels) inside the Dam body. Each tunnel is about 1500 meters deep. The upper gallery contains two 130-SMHRs plus one at the entrance to the gallery, and the southern lower gallery has one 130-SMHR. The fifth 130-SMHR is externally located on the crest of the dam [16].

The 130-SMHR strong motion accelerograph, which combines the 130-01 broad band seismic recorder and an internal low-noise force-balance triaxial accelerometer, was the perfect fit for this application. It provides accurate and timely data and information for seismic events, including their effects on buildings and structures by employing modern monitoring methods and technologies. The 130-SMHR has advanced communications features including transmission control protocol/Internet protocol (TCP/IP) over Ethernet and Asynchronous Serial. An LCD continuously displays state-of-health and status information.

The 130-SMHRs installed inside the dam body are powered by 220VAC mains power. The free field station at the North side of the dam base and dam crest are supplied by a solar energy system. The diagram in Figure (9) illustrates the power and communication equipment at the entrance of the two galleries at the High Dam.

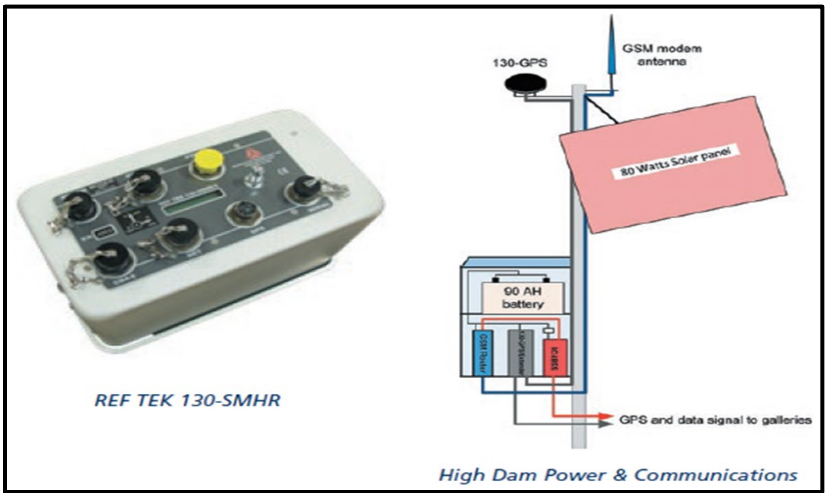

Figure 9: Illustration of the power and communication equipment at the entrance of the two galleries at the High Aswan Dam [16]

In the Old dam, and as this dam does not contain galleries, a total of five 130-SMHRs were installed near the structure; one on bedrock on the ridge of the river; one at the Dam base; three at the Dam crest, with power supplied by a solar energy system. The illustration in Figures (10 and 11) shows the location of the instrumentation.

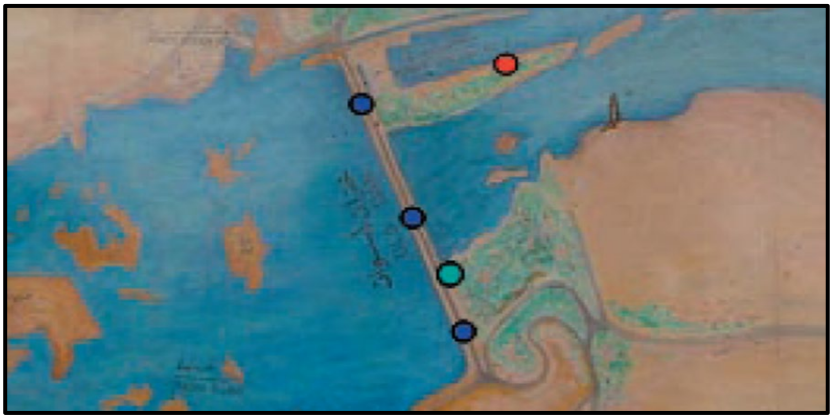

Figure 10: Old Aswan Dam instrumentation location [16]

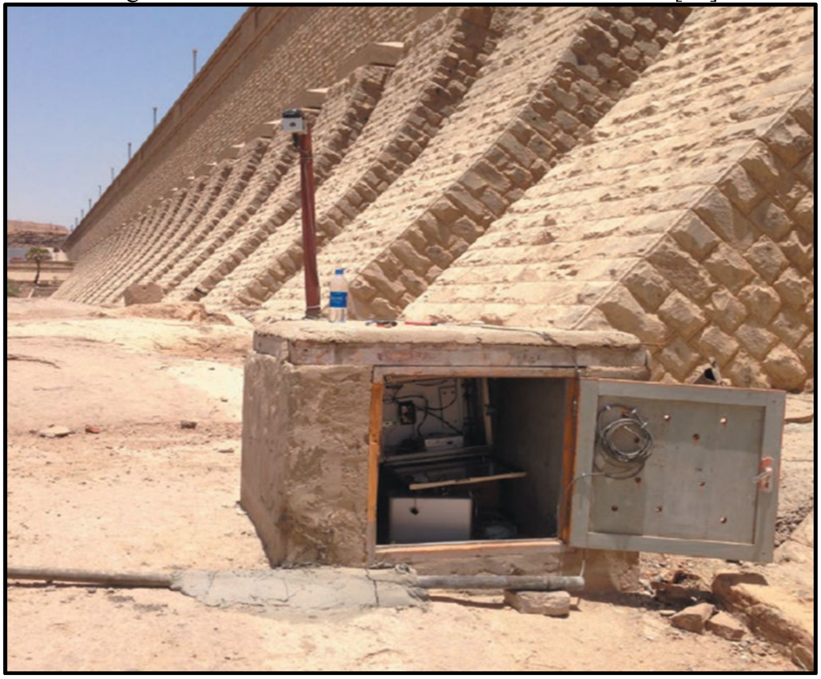

Figure 11: Old Aswan Dam Field Station [16]

As neither internet access nor reliable land lines are available to transmit data from the remote locations inside and outside of the two dams, Global System for Mobile telecommunication technology (GSM) is utilized for real-time data transmission. GSM is a proven technology for both temporary and permanent seismic telemetry network communication media. Net Module Wireless Routers. are used to connect networks and sites Figure (12).

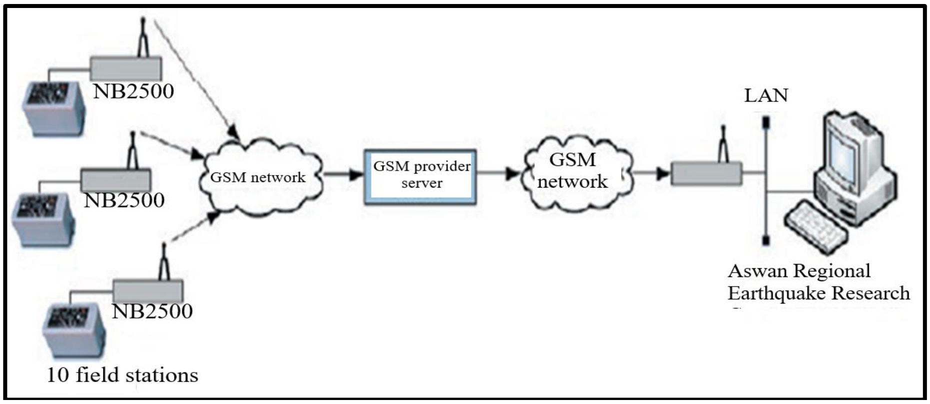

Figure 12: Illustration of GSM Network [16]

All data transmitted from the strong motion instruments in the two dams are received by the established Aswan Regional Earthquake Research Center for further processing.

Data processing equipment supplied to the processing center included two Servers equipped with advanced software for data acquisition, data analysis and data archiving. The acquisition software is capable of downloading data from remote instruments automatically on trigger, and on-demand. This software is also capable of configuring, controlling, synchronizing the 130-SMHRs remotely as well as monitoring the state-of-health. Mission critical computers and peripherals were installed at the processing center, as indicated in Figure (12).

The 130-SMHR has built-in hardware to support IP communications. The applications supported by them are an FTP server, a command server and an RTP protocol client.

Instrumentation of the Aswan dams has provided NRIAG scientists with invaluable information on the structure’s response to seismic activity based on time-lapse observations and early warnings. Using the information, field crews are enabled to accomplish rapid deployment and prioritized inspections to address the need for safe and cost-effective operation of structures [16].

3.2 Enguri Dam Monitoring System, Georgia

A second example of dam monitoring system is cited from the dam monitoring of Enguri1 Dam. This dam, sometimes called Inguri Dam, is one of the highest concrete arch dams in the world which is located on the Enguri Rivers in Jvari, Georgia. It is part of the Enguri hydroelectric power station. The dam is 271.5 m high and 750 m long. Its crest has a length of 728 m and is 10 m thick. The power station is equipped with five Francis type turbines, 275 Mw each. Construction of the dam started in 1971; power station became partially operational in 1978, and it was completed in 1987. Repairs and refurbishment at the Enguri dam took place in 1999, and in 2011 work began to complete the rehabilitation and to ensure safe water flow towards the black sea. The Enguri dam was listed as cultural heritage of Georgia in 2015, Figure (13).

Figure 13: View of Enguri Dam, Georgia

Georgia is situated in the Caucasus, which is one of the most seismically active regions in the Alpine- Himalayan collision belt. Historical analysis shows that it is a region of moderate seismicity and that strong earthquakes have occurred here in the past, including a 7.0 (M) earthquake in the region of Racha in 1991, which killed 270 peoples.

The scope of the Enguri Dam seismic instrumentation project was installing dam monitoring system to record seismic motions and other ambient dynamic activity in order to continuously monitor dam structural safety within the context of a safe operating dam environment [17].

The solution was to install 10 (AC- 63) force balanced accelerometers, 10 (GSR- 18) strong motion recorders, an interconnecting cabling and modem system (GRX- ICC interconnection set), and a central processing system center with processing and reporting software, Figure (14).

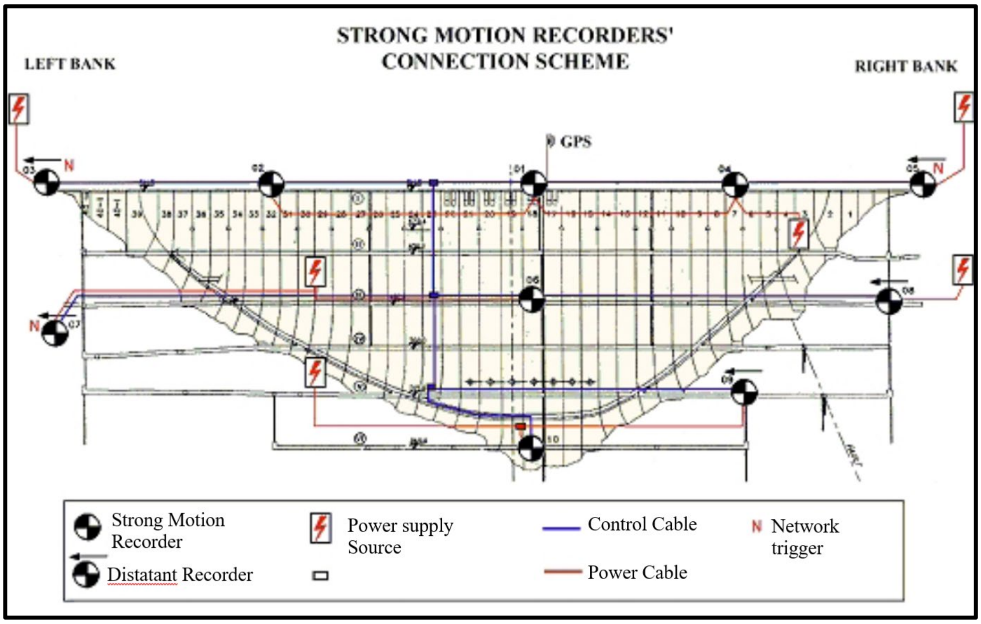

Figure14: Strong Motion Recorders Connection Scheme [17].

Once the data has been processed, it is assessed and compared as the dam behavior against seismic design criteria applicable to dam operation. The project facilitated the development and improvement of dam emergency and safety measuring equipment within the context of increased awareness and contributed to the regional data management systems. Note that the instruments models cited here belong to instrumentation manufacturer and developer GeoSig, Switzerland.

4. Dams Response to PGA During Earthquakes

Damages sustained by dams in response to strong motion acceleration, measured at their sites, depends on type of the dam, whether concrete or earthfill, its height, its design, and the previous seismic history of the dam. As some damages may tend to accumulate due to recurrent events which tend in earthfill dams to change dam material's properties.

The main objective of seismic instrumentation of dams is monitoring their behavior when an earthquake occurs and recoding their response. The other important use is to compare recorded PGAs with those assumed for the design and examine ways of improving dams` safety; if the recorded values exceed the design assumptions.

The intensity and duration of ground vibration that a dam can tolerate without experiencing damage are quite variable. The physical factors that create such variation in dams when subject to earthquake ground vibration are; material used, its density and water content. Not only does physical property variability create problems, but also the parameters that best describe the necessary intensity of ground motion to create structural damage. These parameters include the maximum displacement, maximum particle velocity, maximum acceleration, and the natural frequency of the dam body. In this respect each dam is unique case which means that it should be instrumented as it stands.

Seismic instrumentation is used to record the response of a structure, foundation, and abutments to seismic events. One of the specific applications of the acquired measurements is to furnish data to decide if the structure will continue to function as intended, and for the purpose of evaluating its behavior in such an event verifying design assumptions made for such case. Sustained damages observed in dams after an earthquake may be correlated with the recordings of the seismic instruments installed at or near dam sites. This correlation can help for better assessment of design requirements to resist the ground shacking that may be caused by similar earthquakes in future planned dams. Normally, measured parameters by seismic instruments are the maximum ground acceleration (PGA) which is caused by an earthquake of magnitude (M) whose focus is at known distance from the dam, and moreover, the time history and frequency of such ground vibration.

Ground shaking during an earthquake can cause various kinds of damages. This matter depends on the previously mentioned parameters in addition to the type of the dam itself and the seismic criteria used for its design.

In embankment dams, damage may appear in the form of settlements which can reduce freeboard, and deformations of the side slopes in addition to various types of cracking with various degrees of seriousness and extent leading to increased seepage. Liquefaction of the dam or its foundation materials is another possibility which depends on the type of these materials and their degree of saturation and the intensity of ground shacking.

In concrete dams, earthquake ground vibration can cause cracking of the structure, liquefaction of the foundation, structural movements and deformations, settlements, seepage and piezometric level rise in dam foundations leading to uplift pressure increase.

For the purpose of illustration of the embankment dams' response and possible damages due to measured peak maximum acceleration (PGA), examples are presented of embankment dams and concrete dams are presented in paragraphs (5) and (6) respectively.

5. Embankment Dams Response to actual PGA During Earthquakes

Three examples of embankment dams' response to actual PGA during Earthquakes are presented; two are for rockfill dams, and one for concrete faced rockfill dam. It must be stressed that no dams of the same type will respond in similar manner under seismic ground shacking; as this depends on the characteristics of the earthquake, its focal length and the geology and topography of the terrain, in addition to the design of the dam and the properties of its materials. It is necessary, however, to study as many cases as possible to draw general conclusions of the most probable type of damage to be expected and take necessary precautions.

5.1 Matahina Dam Case (1987), New Zealand

Matahina Dam is a large embankment rockfill dam which was shaken by Edgecumbe earthquake (M 6.3), New Zealand on May 2, 1987. The earthquake had caused a peak ground acceleration of 0.33g at the base of the dam and triggered subsidence or settlement at the crest, revealing and boosting internal erosion. The dam response was recorded by five strong motion accelerometers and a maximum crest level acceleration of 0.42 g was measured. The rockfill at dam crest settled by 100 mm and the dam moved by 250 downstream generating deformation of the dam shoulders.

The earthquake occurred near the town of Edgecumbe. The dam is located on the Rangataiki River about 23 km south of the main shock epicenter and 11 km from the main surface faulting, Figure (15), [18] and [19].

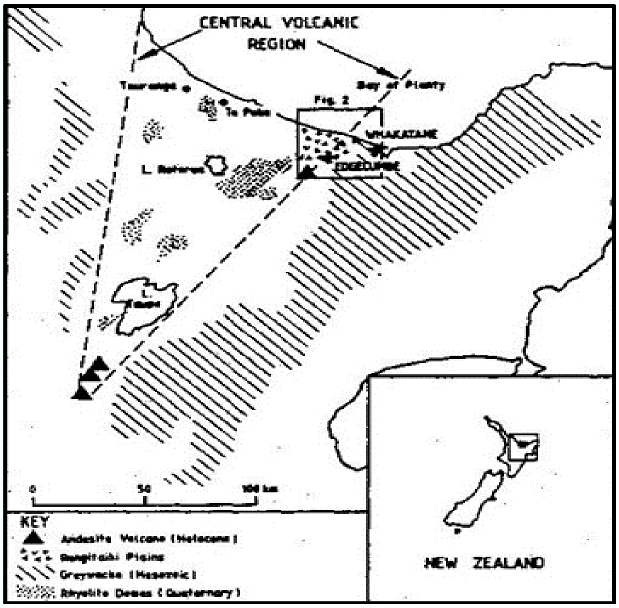

Figure 15: Regional Geology; Matahina Dam [19]

In the two weeks preceding the main event, there were earthquake swarms in the region; these are culminated in the main shock at 0.1hr 42m 34s on 2 March 1987 UT. The epicenter location was 8 km NNW of Edgecumbe, Figure (15), and the focal depth was estimated to be 12 km. The magnitude of the main shock was 6.3.

A foreshock and four aftershocks had magnitudes of over 5.0 with epicenters within a few kilometres of the main shock. The accelerometer at the base of the dam recorded M5.2 foreshock, the M6.3 main shock and the largest aftershock, the M5.5 event 8 minutes after the main shock. The transverse (upstream-downstream) acceleration, velocity and displacement record from the base of the dam and the 5% damped acceleration response spectra are shown in Figure (16a). The El Centro 1940 (N-S) response spectra is shown for comparison Figure (16b).

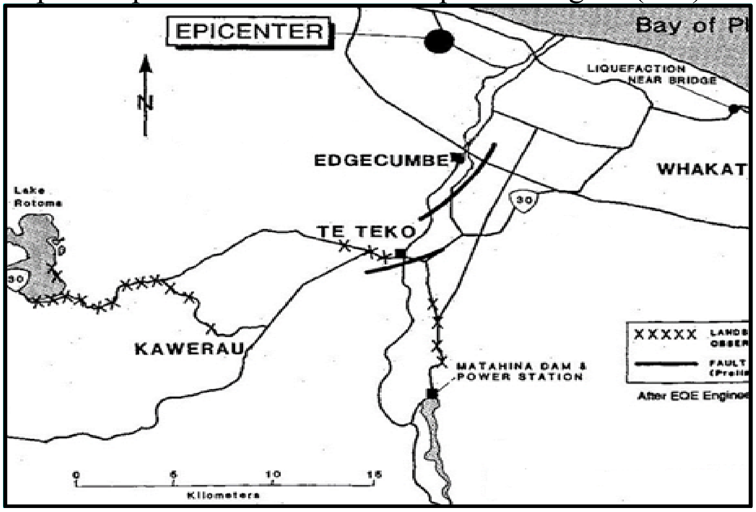

Figure 15: Location of Fault Rupture [19]

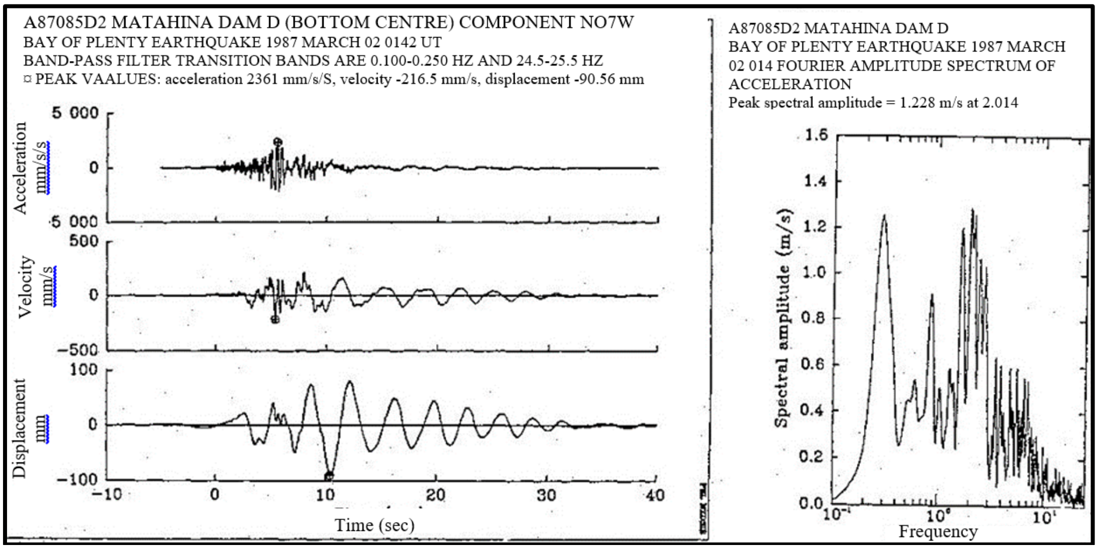

Figure 16a: Acceleration. Velocity and displacement at the base of Matahina Dam [19]

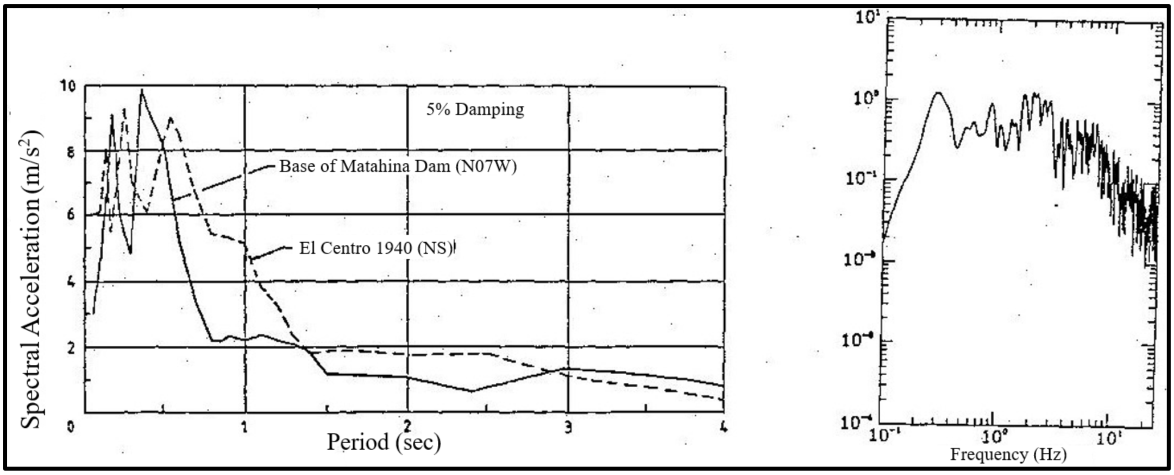

Figure 16b: El Centro earthquake (1940) 5% damped acceleration response spectra (for comparison) [19]

The dam, Figures (17 and18) stands 86 m high above foundation level and has a crest length of 400 m. It has an upstream sloping core of moderate width. The core material is weathered greywacke with a low plasticity gravelly clay grading. The dam shoulders are of hard ignimbrite rockfill compacted by heavy tractor track rolling. The transition zones between the core and shoulders comprise the fines and softer stripping from the ignimbrite rock quarry.

There is a grout curtain forming a partial cutoff within the spur supplemented by two drainage drives. Seepage flows and groundwater levels are monitored.

Under seepage is controlled by a shallow cutoff below the core and a 30 m deep curtain of drain holes which discharge into an extensive drainage blanket. Flow from the drainage blanket is monitored by a weir located in the old river channel downstream of the dam.

The dam instruments include five strong motion accelerometers from which records of the foreshock and main shock were obtained. The extensive surface monument network had been resurveyed three weeks prior to the earthquake.

During lake filling in 1967, core cracking, leakage and internal erosion occurred above a step in the right abutment. High turbid leakage flows were observed at the drainage blanket monitoring weir. An erosion cavity was subsequently located downstream of the core. Repairs comprised a plastic concrete patch on the downstream side of the core backed by granular filter zones. The core was grouted with a cement bentonite mix and the lake refilled without further incident.

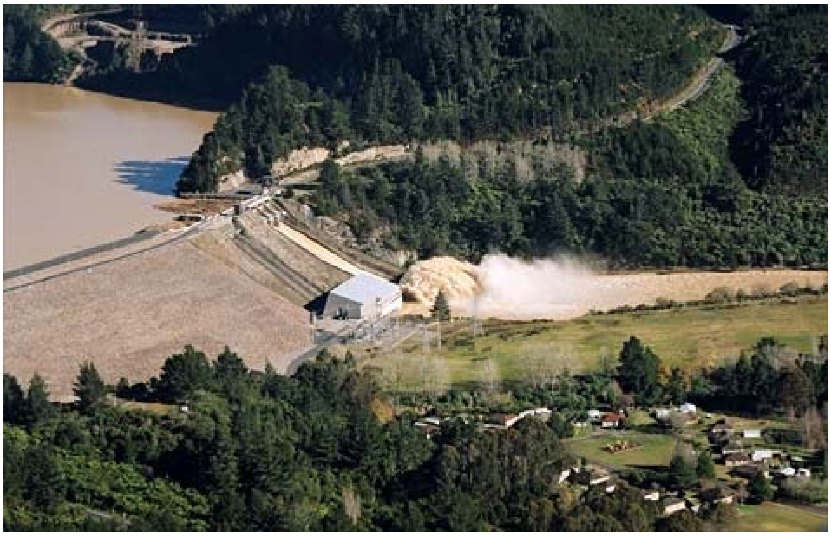

Figure 17: View of Matahina Dam [20]

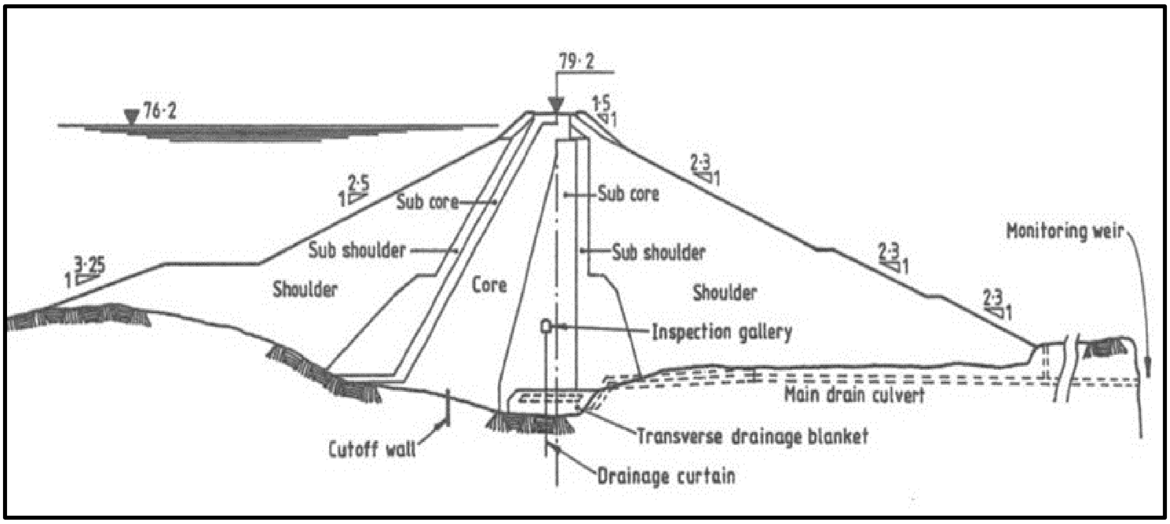

Figure18: Cross section of the Matahina Dam [21]

Detailed inspection following the earthquake showed surface cracking and minor local settlements near the abutments, a turbid and increased drainage flow from the left abutment spur, a minor increase in flow at the drainage blanket weir, settlement and downstream displacement of the crest and large settlements in the upstream rock fill shoulder. Investigating the damages revealed the following:

i) Seepage

It was observed during the period of lake drawdown following the earthquake. Flow from the drainage blanket weir increased from 70 1/min to 630 1/min. Four days after the earthquake flow from the weir ceased and it has flowed only intermittently since then. Investigations concluded that in the periods of low tailwater level flow from the drainage blanket leaks into the groundwater system downstream of the dam. The increase in flow after the earthquake was probably due to the increased tailwater level during drawdown and that increased leakage up to 2000 1/min may not be detected during periods of zero weir flow, and no major dam leakage has occurred as a result of the earthquake.

Seepage flow from the left abutment rock spur which rose to four times the normal flow after the earthquake has continued to slow rise. This trend was continued to be observed and was closely monitored eight months after the earthquake.

ii) Settlements

Measured settlements of survey points are shown in Figure (19), which indicates the very small settlements of the left abutment spur and the much larger settlement of the dam immediately after the earthquake. Given the level of shaking, the settlements are not considered excessive, but they were viewed with concern because of the core cracking and erosion associated with lake filling.

Settlement of the dam continued for several weeks. During this time, the crest settlements were less than the rock fill settlements as would be expected. Measurements from the inspection gallery did not suggest significant foundation settlement, Figure (20). The long term settlement of the left abutment spur was unexpected, Figure (19). The results indicate a fairly uniform settlement without tilting. The settlement caused increased leakage into the powerhouse.

The upstream shoulder settlements were estimated. Settlements of 800 mm were typical. Subsurface sonar was used to check for evidence of under-water slope failure. There were no detectable scarps and it is considered that the settlements are simply the result of earthquake induced compaction of the rockfill.

iii) Displacements

The displacement of the downstream rockfill shoulder is shown in Figure (20). The maximum displacement of 253 mm compares with about 220 mm during the lake filling period.

The ratio of long term crest displacement/settlement (typically 2.5) is similar to that observed during the earthquake.

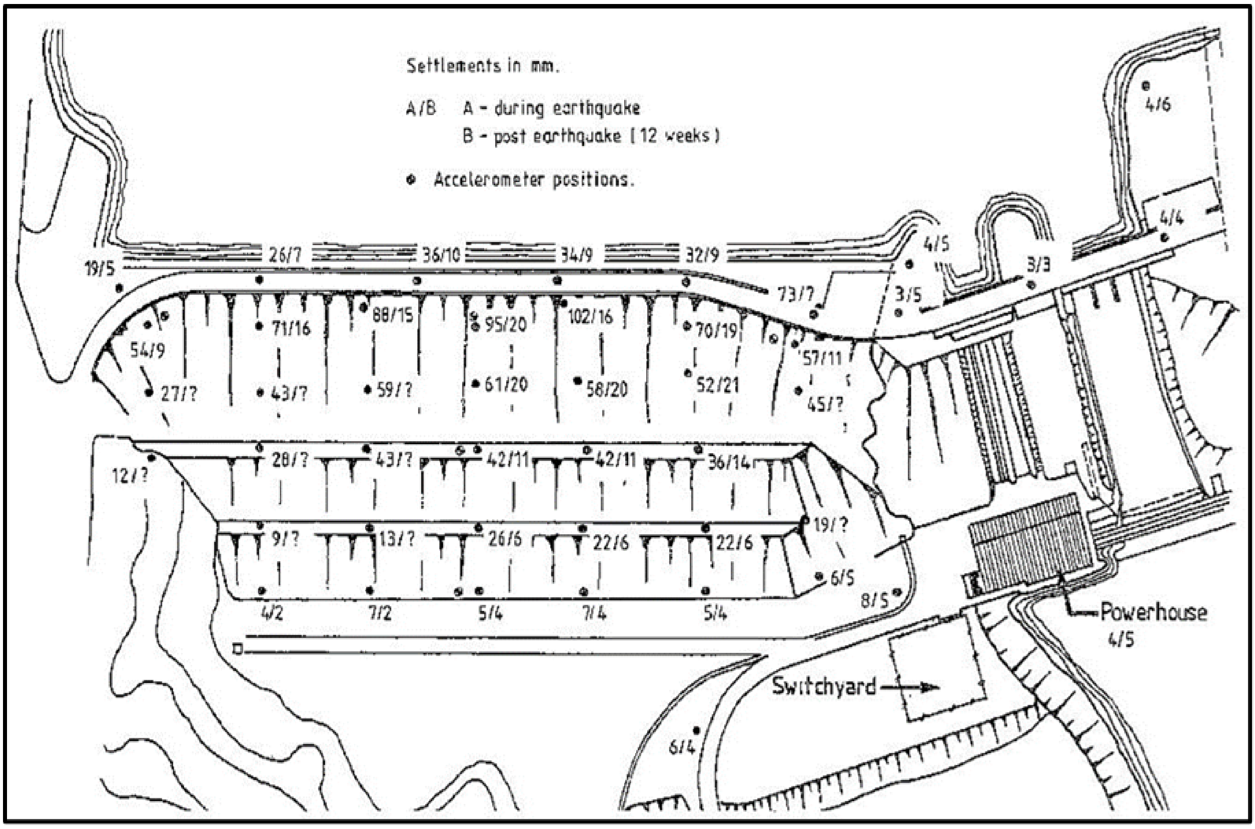

Figure 19: Settlements resulting from earthquake [21]

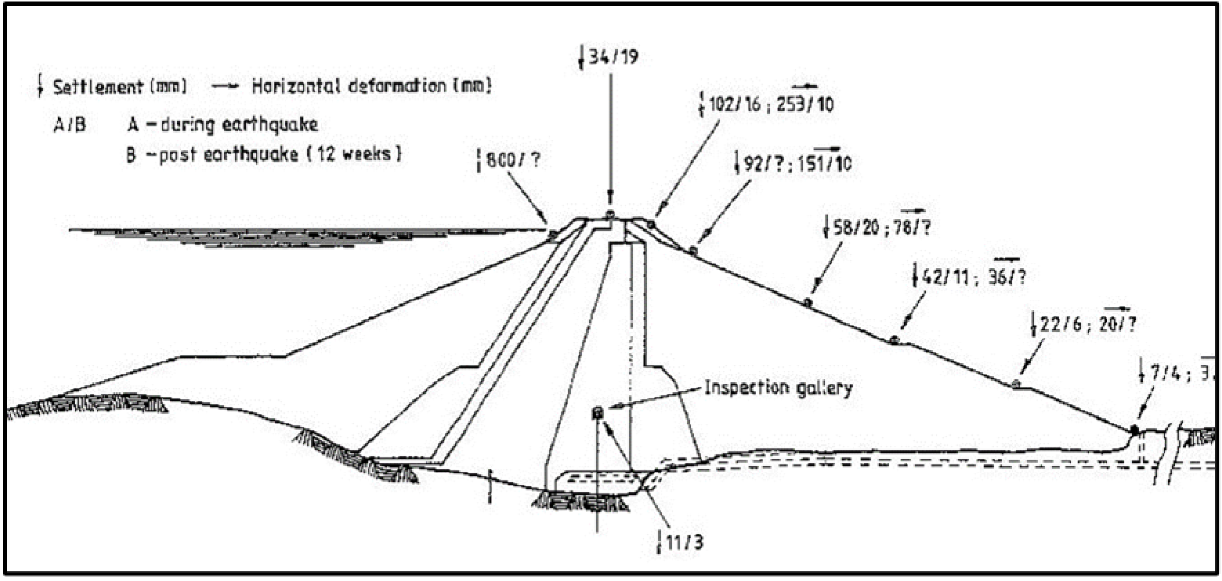

Figure 20: Deformations at the center of the dam [21]

iv) **Left abutment area**

Four inclined drill holes were drilled using air flushing. High water inflows were encountered downstream of the core of the dam. Caving areas were found in two holes. Twelve piezometers were installed and anomalously high pore pressures measured. Permeability's measured from falling head tests in the piezometers are at least 1000 times higher than expected.

The results indicate that core cracking and erosion had occurred similar to that observed in 1967 on the right abutment. It is not known whether the defects were predating or postdate the earthquake. Intensive monitoring indicates that they are stable at present. Remedial measures were proposed and executed.

The dynamic behavior of the dam was studied based on the recordings of the five strong-motion accelerographs of which the dam was equipped with. These recorded the foreshock and the main shock and some of the aftershocks. Three of these instruments were sited across the crest, one at the center of the base of the dam and one at a mid-height rockfill berm. They recorded the peak accelerations shown in Table (1).

Table 1. Summary of the maximum accelerations measured on the transverse dam's centreline [21]

Description		Acceleration (cm^2/s		
Component		Base	Midheight	Crest
Vertical		1378	2018	3247
Horizontal	Maximum	3247	4680	4155
	Transverse	4366	3209	2824
	Longitudinal	4155	3427	2766

The vertical base acceleration is amplified by a factor of over two at crest level. The horizontal crest components are amplified more in the transverse direction than in the longitudinal direction. An unexpected result is that the highest horizontal accelerations (0.48 g) are measured on the rockfill shoulder at the mid height of the dam which was subject to further investigated.

5.2 La Villita Dam Case (1985) Mexico

La Villita Dam is a 60 m high zoned earth dam in Mexico with a crest about 420 m long,constructedon a 70 m thick alluvium layer. The dam was the principal component of a 304 MW multi-purpose hydroelectric, irrigation and flood-control development. It was designed with symmetrical cross-section with a central impervious clay core, well graded filter and transition zones and compacted rock fill shells. The alluvial layer beneath the clay core was grouted below the dam, while there is also a 0.6 m thick concrete cut-off wall to control seepage through the alluvium below the dam. Figure (21a and b) shows a schematic representation of the transverse cross section of the dam and a longitudinal section. Both sections show the location of three functioning strong motion accelerometers (C), (B), and (R).

Upstream and downstream faces slope at 2.5:1, horizontal to vertical. The dam crest is slightly concave toward downstream. La Villita Dam isconstructed on up to70m thick, well-graded alluvial deposits from the Balsas River. The alluvium is composed of boulders, gravels, sands and silts which taper toward the abutments. The abutments consist of layers of andesite and andesitic breccias. A two-foot wide central concrete cutoff wall extends to bedrock across the entire dam foundation

[22].

The dam was subjected to about 60 seconds of strong ground motion during the September 19, 1985 earthquake, which was recorded at the site, which was located at 75 km from the epicenter of the earthquake, and also on the dam.

The dam was well-instrumented. Five strong motion accelerometers, which include AR-240 and SMA-1 instruments, were installed at various locations within the dam and abutments. The dam was also equipped with 21 vertical and horizontal extensometers, 20 inclinometers, three horizontal rows of hydraulic levels and five lines of survey monuments, two on either side of and parallel to the crest, two near the upstream and downstream toes and one at about mid height of the downstream face. Forty-five piezometers, upstream and downstream from the concrete cutoff, monitor the effectiveness of this cutoff.

On September 19, 1985 earthquakes, the accelerometer at the center of the crest of the dam recorded a peak horizontal acceleration of 0.45 g and, on the following day, a peak acceleration of 0.16 g was measured during the strongest aftershock. Peak horizontal bedrock acceleration was recorded at 0.13 g for the main event and 0.04 g for the aforementioned aftershock. Post-earthquake seismological research studies showed that the September 19, 1985 earthquake resulted from two distinct bursts of energy lasting about 16 seconds each and separated by about 25 seconds. This dual rupture mechanism was more conspicuous on records from other strong motion stations closer to the epicenter than from the La Villita instruments. Bedrock records for the main event are shown in Figure (22).

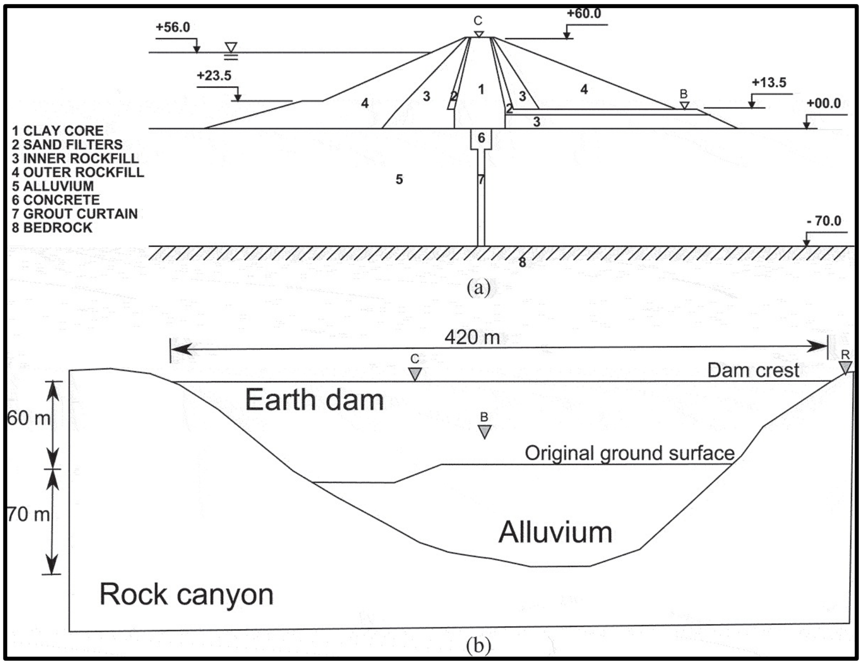

Figure 21: (a) Typical cross section (b) Long section at La Villita Dam, showing location of the strong motion accelerometers (C), (B), and (R) [23]

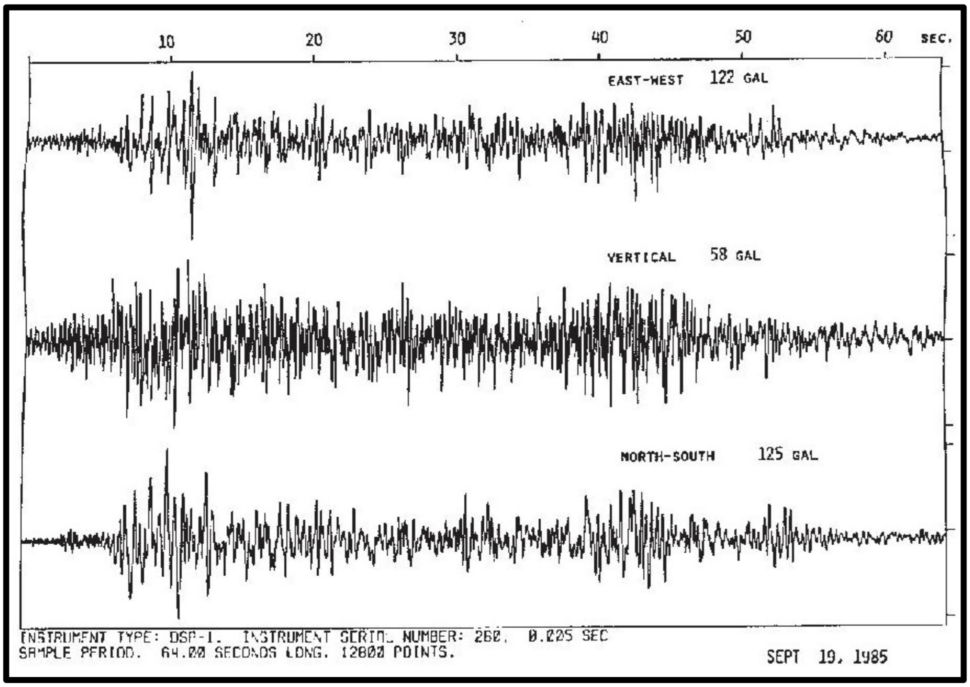

Figure 22: Bedrock Acceleration Records [22]

Damages to the dam was noticeable on the dam in the form of cracking, settlement and displacements, as follows:

i) **Cracking:**
Two main systems of longitudinal cracks developed at the crest of La Villita Dam, parallel to its axis, some 16 feet away from the crest edges. These cracks formed along the buried shoulders of the central core and most likely resulted from differential settlement between the core and the adjacent filter zones. A 260- foot-long crack, 1/4 to 2 inches wide at the surface, formed along the upstream side of the dam crest. Vertical offsets of 2 to 4 inches occurred between the lips of the crack, the upstream side settling the most. On the downstream side, another major crack system appeared, about 1,000 feet long, 0.4 to 0.6-inch-wide, with vertical offsets ranging from 0.5 to 0.8 inch, the side toward the face of the dam being downthrown. Several other longitudinal cracks, up to two inches wide, but less extensive than the two principal crack systems, were also found. The most significant cracks did not reach the impervious core zone and were found to disappear below two-feet depth, except near the right abutment, where one of the cracks was delineated as a closed fissure through a clay lens embedded at about three-foot depth within the filter sands, Figures (23, 24).

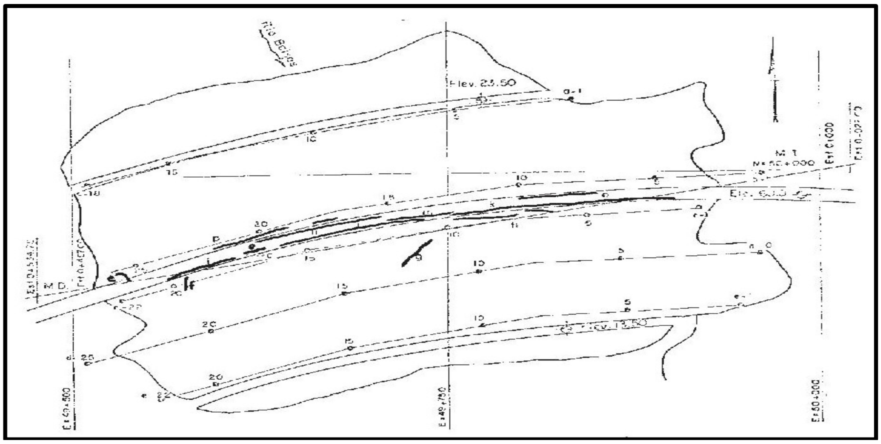

Figure 23: Crack Locations on La Villita Dam [22]

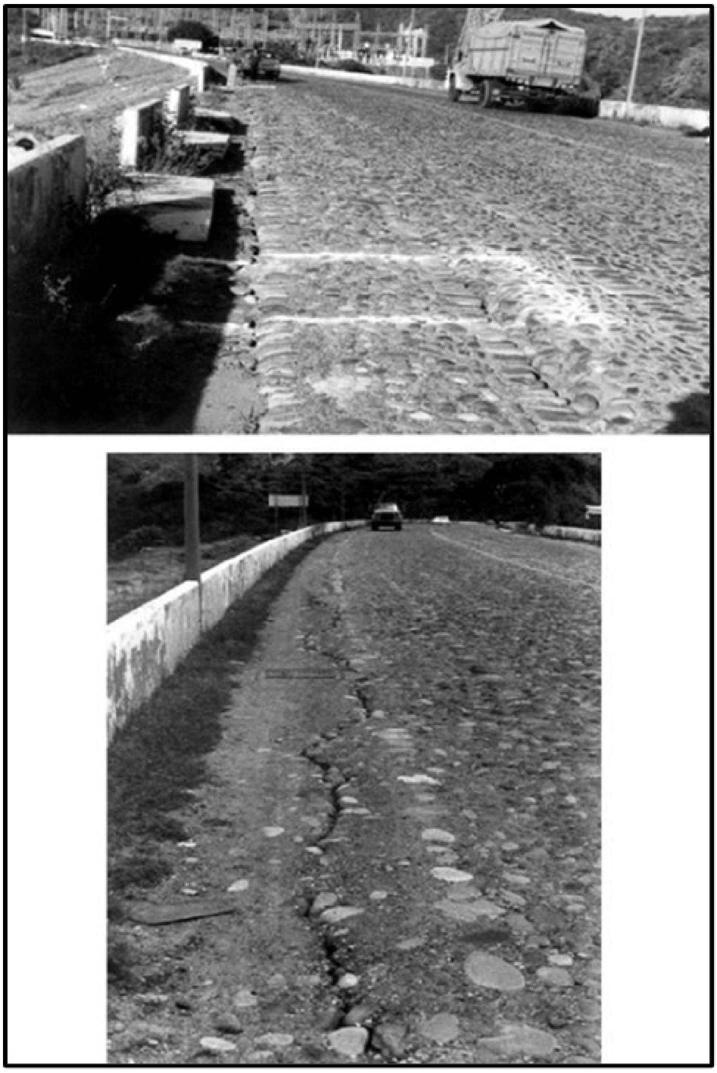

Figure 24: Photographs showing longitudinal cracking at dam crest [22]

ii) Settlements

Post-earthquake surveys showed that, in its central part, the dam settled between 20 and 32 cm on the upstream side and between 9.1 and 22 cm decreased in magnitude to near zero toward the abutments and seemed to be evenly distributed within the dam cross-section, rather than associated with distinct surfaces.

Figure (25) gives the accumulated settlement of the dam due to the earthquakes to which the dam was subjected in previous years. Earthquake-induced settlements have been found to exceed static postconstruction settlements and appear to increase in magnitude from one earthquake to the other, perhaps indicating a change in stiffness of the dam materials or a slow, cumulative, deterioration of part of the embankment. Inclinometer records confirmed that permanent deformations decreased in magnitude from crest to bottom of the embankment and did not involve the foundation materials.

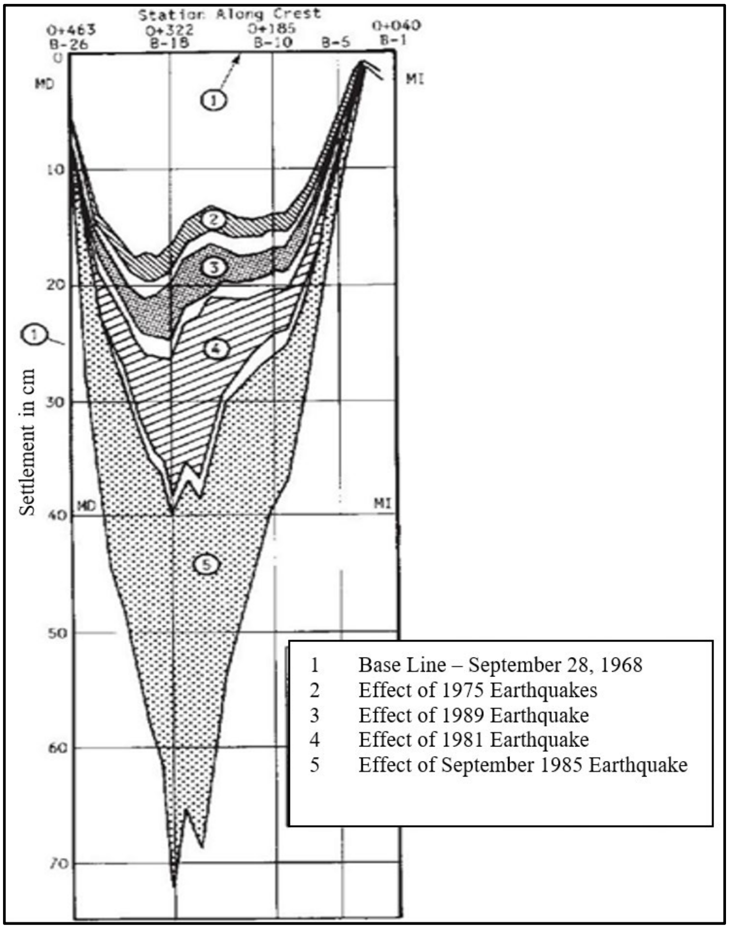

Figure 25: Historical Crest Settlements record showing crest settlements from 1968 to 1985. [22]

Settlements were particularly noticeable at several piezometer locations, where the piezometer tubes which extend down 80m to deep within the embankment remained in place, while their protective concrete boxes settled along with the face of the dam, Figure (26).

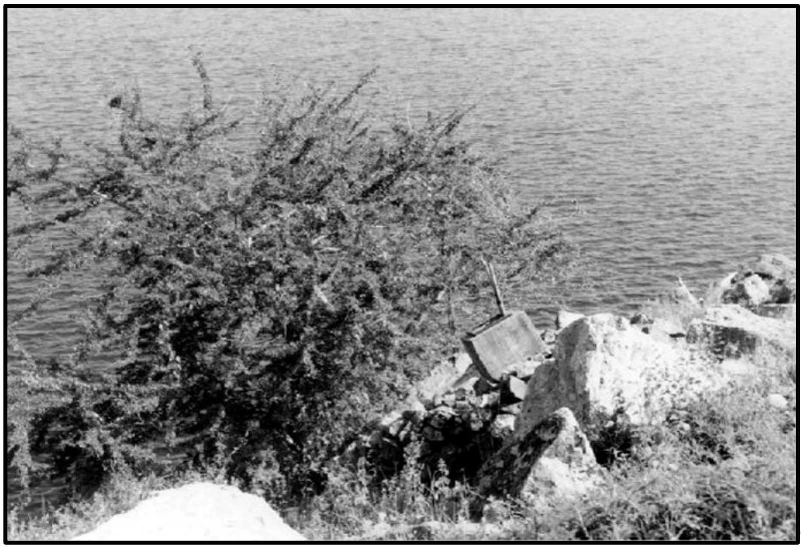

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Figure 26: Settlement at Piezometer box [22]

iii) Displacements

The downstream half of the dam moved horizontally up to 10 cm in the downstream direction and the upstream half up to 16.5 cm in the upstream direction. Downstream horizontal displacements were somewhat irregular, although generally more symmetrical with respect to the center of the dam than the upstream displacements.

These observed damages correlated well with the instrumentation and strong motion records. Survey monuments, inclinometers and extensometers were essential to provide detailed information on the earthquake-induced deformations of La Villita Dam. Of particular interest was the fact that the dam had previously been shaken by several significant earthquakes in the 12 years that preceded the 1985 event.

The dam was well instrumented, especially for strong motion accelerometers which proved their significant value by the records they provided; giving full picture of the dam response under large earthquakes.

It is interesting to note that the 148 m El Infiernillo earth core rockfill dam, which is located in the same area as La Villita Dam was also shaken by the same 1985 (M 8.1) earthquake and the sequence of the five closely spaced events since 1975. The dam suffered also cracking and settlement problems, but the deformations remained small and consistent from one event to the next.

5.3 Cogoti Dam Case (1943), Chile

This dam is a concrete face roc kfill dam built in 1938, it is located about 47 miles (75 km) from the City of Ovalle, Chile. The dam site is situated downstream from the confluence of the Pama and Cogoti rivers, and in a deep gorge naturally carved by the Cogoti River. Cogoti Dam, shown in plan and cross section in Figures (27 and 28), has a maximum height of 85 m, a crest length of 160 m. The upstream slope averages 1.4H : 1V and the downstream slope is about 1.5H :1V .The dam is primarily used for irrigation purposes and impounds a reservoir of 148 million cubic meters capacity [24]

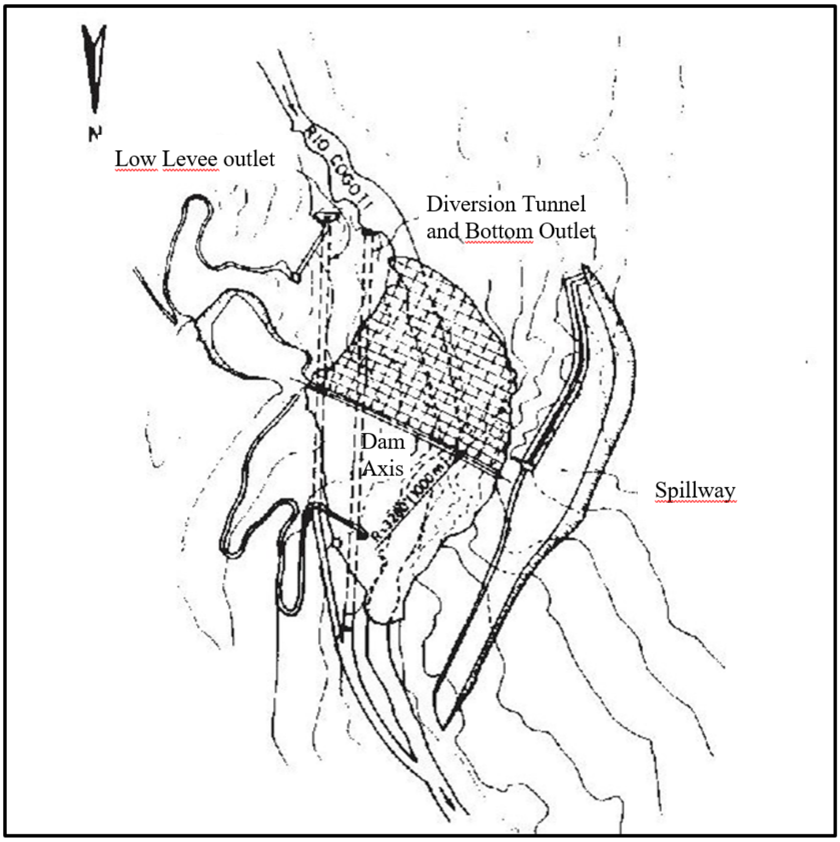

Figure 27: Site plan and Layout of Cogoti Dam [24]

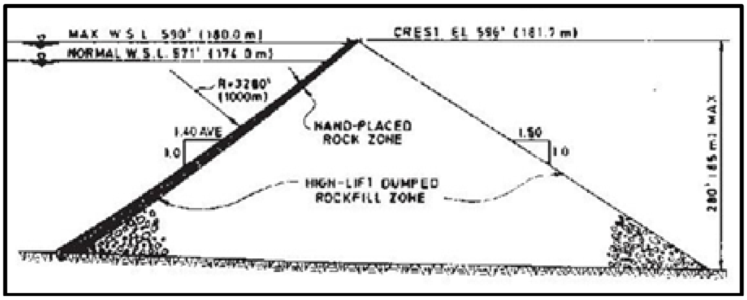

Figure 28: Typical cross section of Cogoti Dam [24]

Local rocks, which consists primarily of andesitic breccia, was used for construction. The main rock fill zone was started by blasting some of the abutment rocks and allowing the blasted rock fragments to fall freely on the foundation. Following completion of the required abutment excavation, rockfill was dumped in lifts as thick as could be practical, and without mechanical compaction or sluicing.

The flexible, impervious, segmented reinforced concrete face was placed on a 2 m thick bedding zone of hand-placed, small-size, rock. It was designed as individually formed slabs, of 10 x 10 m average size, with a thickness tapering from 80 cm at the upstream toe to 20 cm at the crest of the dam. Horizontal and vertical joints with 60 cm wide copper water stops and rivets were provided. The spacing and bar sizes of the steel reinforcement vary as a function of elevation along the dam face, starting with a double curtain of 25mm bars at 30 cm pacing near the toe and ending with a single curtain of 18mm bars at 20 cm spacing at the crest.

The spillway is an ungated channel with a reinforced concrete side-channel having broad crested weir control, and was excavated in the left abutment rocks. It has a design capacity of 4984 m3/s.

On April 6, 1943, a large earthquake (M 7.9) occurred approximately 200 km north of the City of Santiago. The earthquake centered about 95 km from the Cogoti Dam site; peak ground acceleration at the site was estimated to be about 0.19 g. Substantial settlement of the dam was observed as a result of this earthquake.

The April 6, 1943, Illapel Earthquake destroyed most of the towns of Combarbala, Ovalle and Illapel, about 200 km north of the City of Santiago. Damage was reported in a wide area, some including the City of Santiago. However, few references, and none of these technical reports, describe this earthquake. Presumably, this is because the affected onshore area is mountainous, was sparsely populated and was probably considered of minor economic importance in 1943.

The shock was; however, felt as far away as Buenos Aires, Argentina, where dishes were broken and ink spilled from ink wells. Damage extended throughout the province of Coquimbo. A copper mine tailings dam collapsed near the City of Ovalle, killing five persons. Total reported lives lost were eleven. The epicenter was determined to be offshore, directly across the mouth of the Limari River. Earlier magnitude estimates were as high as M 8.3, but were subsequently lowered to a maximum of M 7.9. Many aftershocks were felt during the week that followed the earthquake.

The Illapel Earthquake was centered about 95 km from Cogoti Dam. An intensity IX on the Rossi-Forel scale was reported at the dam site. The reservoir is believed to believed to be at its normal operating level at the time of occurrence of the earthquake. The principal observed effect on Cogoti Dam was to produce an instantaneous settlement of up to 41cm. Settlement occurred throughout the length of the crest, and the extreme upper part of the concrete face slab was exposed from the downstream side, as quoted in an internal report by Empresa Nacional de Electricidad S.A., Santiago, Chile (1972). It is of interest to note that the maximum earthquake induced settlement was about equal to that observed 4.5 years at the end of construction. The point where this settlement was measured was near the center of the crest, where the dam height is about 63 m. This was not the highest dam section, which was located close to the right abutment. The settlement at the maximum dam height was less, presumably because of a restraining effect due to the nearby presence of the very steep abutment. Minor rockslides also occurred along the downstream slope of the dam.

Leakage had been observed at Cogoti Dam since the reservoir's first filling in 1939. Intermittent records have been kept over the years, which indicate leakage to be directly related to the elevation of the reservoir and probably coming through the abutment or foundation, rather than the dam itself. No significant increase in dam leakage was observed as a result of the 1943 earthquake. No face cracks were caused by the earthquake. Yearly settlement and leakage data at Cogoti Dam are presented in Figures (29 and 30).

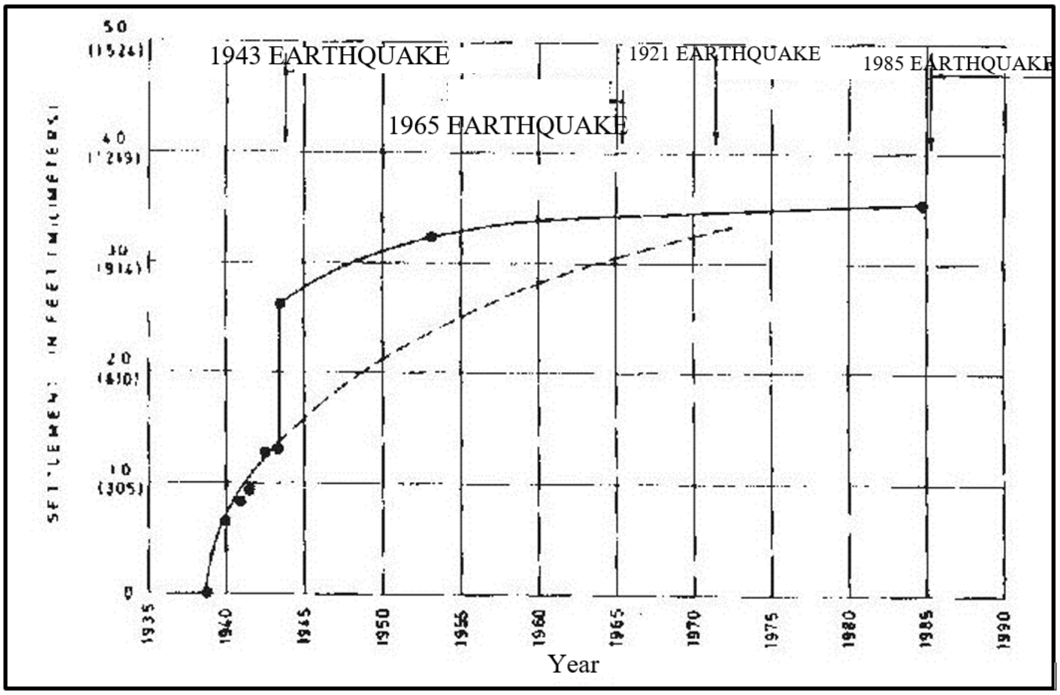

Figure 29: Crest Settlement Curve [24]

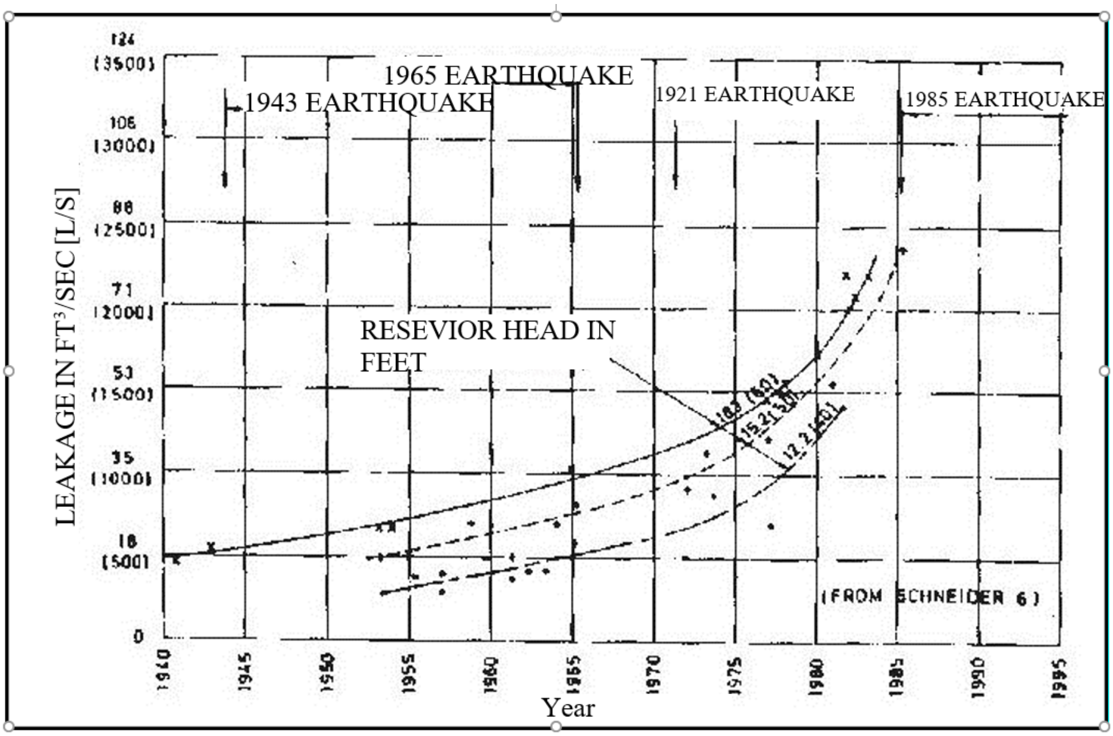

Figure 30: Leakage at Cogoti Dam [24]

The dam has continued to settle after the 1943 earthquake. Interestingly, it was shaken again by three significant, although considerably more remote earthquakes: in 1965 La Ligue Earthquake, M 7.1; in 1971 Papudo-Zapallar Earthquake, M 7.5; and in 1985 Llolleo-Algarrobo Earthquake, M 7.7. These more recent events; however, were centered at distances of more than 165 km from the dam and did not induce any noticeable settlement. Yet, in 1971, even though the reservoir was empty at the time of occurrence of that earthquake, the Papudo-Zapallar Earthquake caused longitudinal cracking at the dam crest and dislodged some rocks along the downstream slope.

Cogoti Dam was not instrumented at the time of the 1943 earthquake, nor were accelerometers installed that could have recorded the subsequent earthquakes. Using an attenuation equation primarily developed from Chilean earthquake data, the peak ground acceleration (PGA) induced at the Cogoti Dam site by the Illapel Earthquake was estimated to be 0.19 g. Peak ground accelerations generated by the subsequent earthquakes were probably less than 0.05 g; therefore, that noticeable settlements were unlikely to occur under such moderate shaking conditions.

Although significant settlement occurred due to Illapel Earthquake, the dam performed extremely well and no seismic damage was observed to the concrete face. Cogoti Dam's performance substantiates the generally accepted belief that concrete face rockfill dams have an excellent inherent capacity to withstand substantial earthquake motion without experiencing significant damage. Although Cogoti Dam's leakage has increased over the years, this has been related to aging and spalling of the concrete and joint squeezing, not to the 1943 Illapel Earthquake nor to any of the subsequent earthquakes to which the dam was exposed.[24].

Leakage increased from 200l/sec to 1400l/sec after the earthquake and, after repairing it decreased to 400l/sec.

On 14 October 1997, the magnitude 6.8 Punitaqui Earthquake with epicentral distance of 20 km and focal depth of 30 km produced a PGA of 0.19 g and a crest settlement of 15 cm. No damage was reported from the concrete face, but some cracking did occur at the downstream face of the dam, Figure (31).

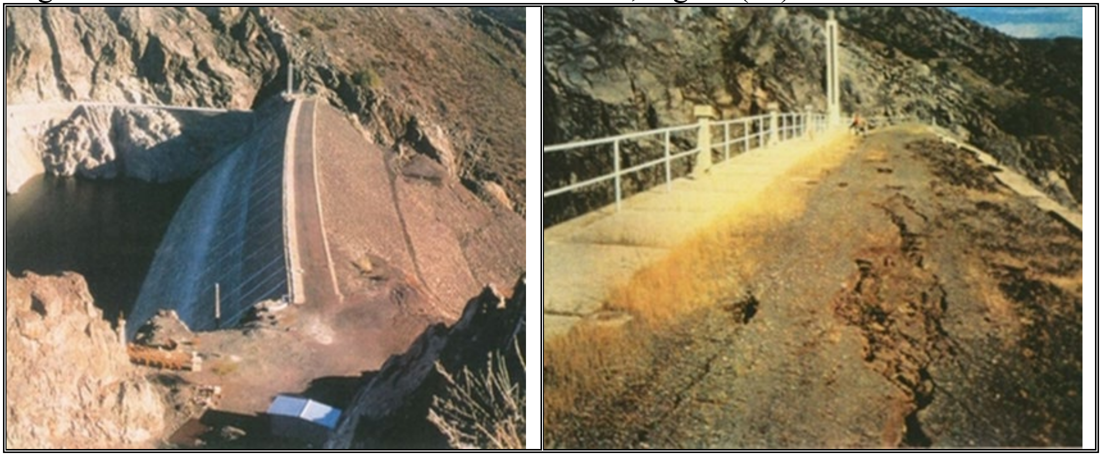

Figure 31: (left) General view of the dam, (Right) a close up view of the dam crest showing longitudinal cracking [25]

The absence of strong motion accelerometer at this dam site is regrettable; as valuable information on the actual response of the dam could have been obtained from the recordings of PGAs acting on dam; therefore, dynamic analysis of the dam was not performed [25].

Moreover, the near-field response of the concrete slab facing still needs to be tested both in experiments and during a strong seismic event. An example is the 85 m high Cogoti CFR Dam in Chile [26].

6. Concrete Dams Response to observed PGAs During Earthquakes

From the recorded response of concrete dams to earthquake ground shacking, it can be seen that these dams generally fare better than embankment dams under seismic loads. Large number of concrete dams have been shaken by earthquakes close to dam sites, but only few have suffered major damages. In fact, during the last fifty years, many such dams experienced peak horizontal ground accelerations (PHGA) greater than 0.3 g. The most severely shaken dams included all principal types of concrete structures: arch, multiple arch, gravity buttress and RCC dams. Generally, concrete dams had performed extremely well when subjected to earthquake motions, even when shaken by forces far in excess of their design loading. It has become apparent that the most significant factor in determining the response of concrete dams is the PHGA and probably the spectral acceleration at the natural frequency of the dam.

No significant damage has ever been suffered by an arch dam, although three such structures have historically experienced substantial ground motions. Concrete buttress dams when subjected to severe shaking have developed horizontal cracks at the elevation high in the dam where the downstream buttresses intersect the vertical “chimney” section. This is an area where the stiffness of the concrete structures significantly changes. Roller-compacted concrete dams; Shapai arch Dam in China and Miyatoko Dam in Japan, performed no differently to date than a dam built of conventionally placed concrete despite concern by some of less strength at the many lift joints.

The only recorded case of concrete dam failure is the case of Shih Kang gravity Dam (Taiwan) which was completed in 1977. The dam suffered of vertical left side displacement of 36 ft, a vertical right side displacement of dam 7 ft and a diagonal offset of 23 ft. The line of fault produced by Chi- Chi Earthquake, M 7.6 Sept 21, 1999 passed exactly under the dam and peak Acceleration Recorded 0.3 miles away Horizontal 0.51 g, Vertical 0.53 g. The dam suffered of cracking along the ogee crest, along lift lines at point of changes in geometry and the dam separated from foundation. This case; however, may be considered the most extreme case due to the fact that the distance of the dam from the line of fault was zero, and the PGAs were so high, Figure (32) [27].

The location of the dam site is shown in Figure (33) [28].

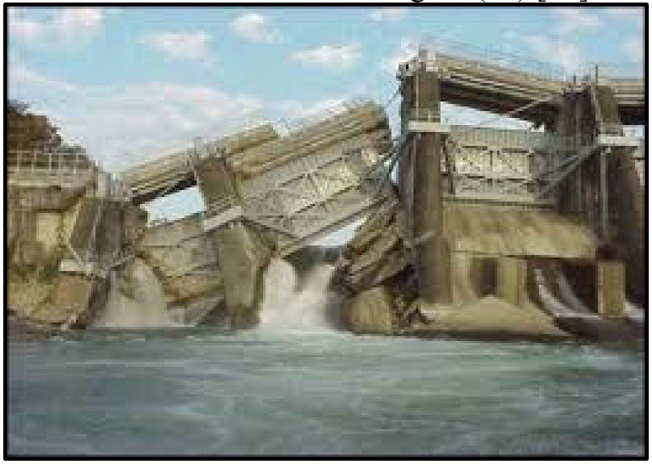

Figure 32: Shih Kang Dam gravity dam immediately after the Chi- Chi Earthquake on September 12th, 1999 [27].

The availability nowadays of high quality strong motion records derived from refined strong motion accelerometers, at or close to dam site, has enhanced our knowledge of dams’ behavior in response to earthquakes. Correlation of the type and magnitude of damage with the measured PGAs at dam sites is possible now which helps in better designs of future dams.

Since the 1979, many large magnitude earthquakes have occurred as expected. With a greater number of higher quality strong motion instruments located at or near dams, our base of knowledge of the magnitude of shaking to which concrete dams have been subjected has increased. Thus, the performance of severely shaken concrete dams has increased and this knowledge can be applied in a positive and beneficial manner to the design of future dams [29].

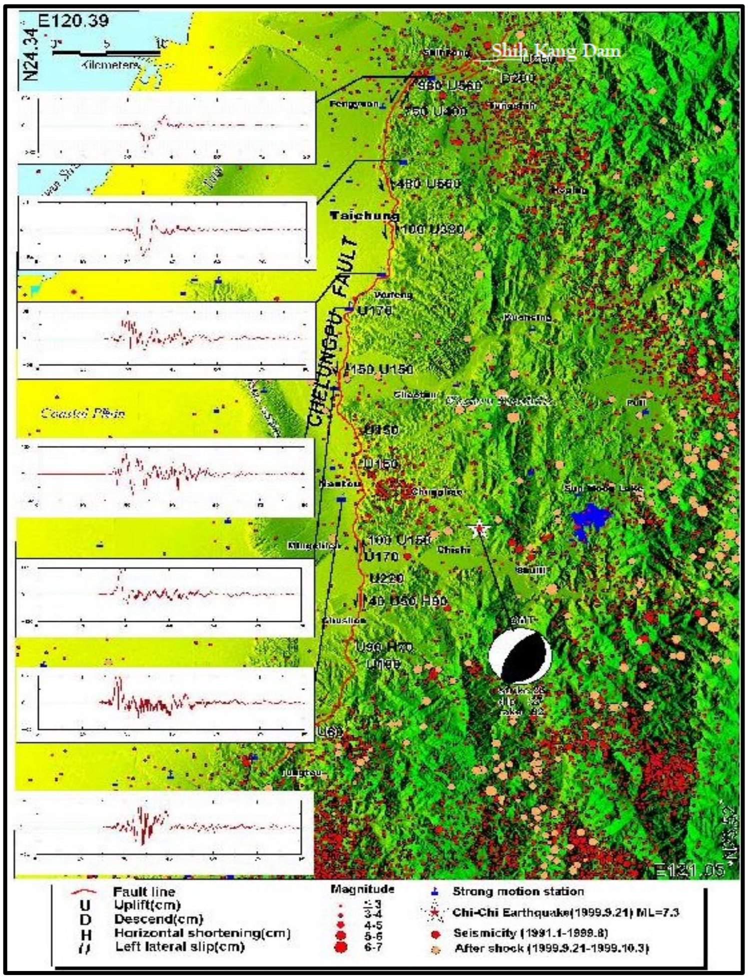

Figure 33: The Chelungpu fault, and the epicenter of the Chi- Chi Earthquake Also shown are background seismicity, strong aftershocks, E- W component velocity forms along the fault line [28]

In a table annexed to the same paper [29], but published in the International Water Power and Dam Construction Magazine the response of (19) dams during earthquakes was described [30]. The table included many cases of concrete dams that were shaken by Peak ground acceleration (PGA) of 0.3g or more (measured or estimated). This table is reproduced in an abridged form in Table (2) which contains selected and representative cases only. More details are given on these dams in reference [27].

Table 2: Concrete Dams Response in Earthquakes for (PHGA > 0.3 g) (1 January, 2000) [30].

Legend: b=base, c=Crest, est=Estimate, disp.=Displacement, rt abut=Right Abutment, Ht= Height, V=Vertical, M=Magnitude (ML or mb for less than 6.5 and MS above 6.5 [31]), Surf=Surface, aft=Aftershock, PHGA=Peak horizontal ground acceleration

Dam (completed)	Country	Ht (m)	Type of Dam	EQ Name and date	M	PHGA (g) [31]		Remarks
Koyna (1963)	India	(103)	Gravity	Koyna 11 Dec 1967	1.8 (3)	6.5	0.63	Cracks in both faces
Mingtan (1990)	Taiwan	(82)	Gravity	Chi- Chi 21 Sep 1999	7.5 (12)	7.6	0.4 to 0.5 (est)	No damage
Kasho (1989	Japan	(46.4)	Gravity	Western Tottori 6 Oct 2000	1.9 or 5 (3 or 8)	7.3	0.54 b 2.09 c	Cracks in Control Building at crest
Miyatoko RCC (1993)	Japan	(48)	RCC	Tohoku 11 Mar 2011	84 (135)	9.0	> 0.7	No Damage
Takou (2007)	Japan	(77)	Gravity	Tohoku 11 Mar 2011 [4]	68 (109)	9.0	> 0.4	Cracking of gatehouse walls at crest
Bear Valley (1912, 1988)	USA	(28)	Multiple arch modified to gravity dam in 1988	Landers 28 Jun 1992, Big Bear 29 Jun 1992	28 (45), 9 (14.5)	7.4, 6.6	0.57	Multiple arch modified to gravity dam in 1988. No damage, except slight displacement of crest bridge girders
Gibraltar (1920, 1990)	USA	(52)	RCC	Santa Barbara 29 Jun 1925	6.5 and 6.8	6.3	> 0.3 (est)	No damage. Modified in 1990 with RCC
Rapel (1968)	Chile	(110)	Arch	Santiago 3 Mar	28 (45)	7.8	0.31	Damage to spillway and

				1985			near dam	intake tower, dam performed well
Techi (1974)	Taiwan	(185)	Arch	Chi-Chi 21 Sept. 1999		7.6	0.5 b 0.86 c	Local cracking of curb at dam crest
Shapai RCC (2003)	China	(132)	RCC	Wechuan 12 May 2008	7.8 (12)	8.0	> 0.5 (est)	No damage
Sefid Rud (1962)	Iran	(106)	Buttress	Manjil 21 Jun 1990	Near dam site	7.7	0.714 (est)	Horiz, cracks near crest, minor disp, of blocks

The analysis of the seismic response of a concrete dam is a complex problem in which the accurate representation of the material's behavior requires some form of nonlinear model, especially if the concrete material is subjected to significant tensile stress demands. In case of severe ground motions, considerable cracking is likely to develop across extensive regions of the dam, particularly at the dam heel and in the vicinity of abrupt changes in geometry. Therefore, the proper consideration of this nonlinear phenomenon and its consequences on the dynamic response of the system become critically important for a rigorous seismic evaluation. The actual post-cracking behavior and the ultimate capacity of existing concrete dams can only be determined by performing the corresponding nonlinear dynamic analyses [32]. This analysis requires the seismic spectra measured on dam sites or nearby sites as an input data in addition to material properties assumed for linear and nonlinear dynamic analyses.

7. Summary Points and Conclusions

7.1 Since the late 1980s, the need for rational method of design for moderate or significate risk dams has raised the need for using seismic monitoring instrumentation in dam sites. These were required for recording peak ground acceleration and its variation with time during the event, together with the three dimensional displacements in dam body and deformations of its slopes. Recording the response of pore pressure devices accompanying the measured PGAs was also done. The development of equipment for transmitting these information and observation to observation centers helped relaying them from remote sites. Nearly, all seismic instruments in use today utilize servo-accelerometers that have the ability to measure motion in a single horizontal, vertical, or transverse planes. Most of the devices are considered "strong motion" instruments that record significant movements.

7.2 Seismic instrumentation of dams and reservoirs' sites is accepted nowadays not as luxury items for research work, but mainly to understand significant seismic hazards facing existing dams in seismic areas and even desirable in traditionally nonseismic areas. With the advent of digital seismic equipment, it can now be an integral part of dam safety monitoring works. The digital earthquake data can be gathered by site personnel and remote control centers by use of computers. When digital instruments are installed with modem and communication means, then remote access from several offices is made available. Seismic instruments' recordings taken from existing dam sites help also in the safe design of new dams in seismically similar regions.

7.3 The increased awareness of the importance of seismic instrumentation in dams have led to installing them in large number in strategic dams. Examples are given of two cases only for demonstration, but many more are given in manufacturers literature. Many high caliber manufactures are available in the world nowadays; their literature shows the degree of advancement in instruments manufacturing, software development and progress in data transfer equipment. Seismic instrumentations are costly to install, run and maintain, but all agree on their extreme value which justifies their use in dams and make their installation highly recommendable.

7.4 Normally, recorded parameters are the maximum ground acceleration (PGA) caused by an earthquake of magnitude (M) whose focus is at known distance from the dam. Damage sustained by dams after an earthquake can be matched with the recordings of the seismic instruments used at dam sites. Apart from PGAs, other obtained recordings are the time history and frequency of the event so a full picture is formed. The correlation of damage with the seismic records gives good guide in selecting the most suitable type of a future dam in any seismic region. Seismic measurements show that concrete dams are less affected by earthquakes than embankment dams under similar seismic loading conditions, and even that rockfill dams have better performance than earthfill dams. We have tried to explain this by presenting many case histories of dams in terms of increased seepage, settlement, displacement and cracking. From these and many other documented cases, there are lessons to be learnt by designers of the best ways to handle any new design or the best rehabilitating and upgrading program needed for damaged existing dams.

7.5 It was possible from seismic measurements and dynamic analyses of affected dams to discover the reasons why concrete dams have performed well and invariably better than that predicted by design or analysis when shaken by an earthquake. These reasons may be:

i) the redundancy of the damaged structure to redistribute load,
ii) the duration of strong shaking being too short to cause sizable damage,
iii) the increase in the tensile strength of the concrete during dynamic loading that increases resiliency,
iv) increase in the damping that reduces the seismic impact on the dam,

v) reduced seismic impact because the natural frequency of the dam does not match the frequency of the earthquake, and

vi) the three-dimensional effects of canyon confinement or dam geometry (curvature) that help prevent failure [30].

7.6 It may be seen from the foregoing that, seismic instrumentation for measuring and recording peak ground acceleration (PGA) and related parameters at dams' sites during earthquake adds to dam safety precautions. They are as valuable to dam safety as other conventional dam instrumentation, their use is therefore highly desirable and recommendable.

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Dam Safety: Monitoring of Tailings Dams and Safety Reviews

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Abstract

The awareness to tailings dam safety monitoring and reviews has increased by the catastrophes resulting from failures of such dams worsened by increasing tailings waste and construction of larger dams. The losses born by the mining industry from high costs of compensations and environmental rehabilitation work have brought this matter into focus. In the present article. The need for safety monitoring programs of tailings dam is highlighted and mode of failures and factors leading to them are described. Basic principles of such programs are investigated with all phenomena needing observation described and their impacts explained. As in conventional dams this work is carried out by visual inspections and use of similar methods and instruments. In similar manners in both types of dams' observation and measurements are done for measuring seepage water quantity and quality, phreatic surface level and pore pressure and total earth pressure values in addition to deformation measurements; and all are done by similar devices and methods such as weirs, piezometers, inclinometers, settlement plates and geodetic surveying. Basic differences between safety monitoring systems of the two types of dam, however, are presented in a tabular form. The continuity of safety monitoring of tailings dams is emphasized not only during the long construction phase but also after that in the abandonment and closure phase which can last indefinitely in order to watch for possible adverse effects on the environment and ecosystem due to the winds eroding and carrying of poisonous tailings contents, in addition to contaminated seepage water entering surface water streams and ground water. Justifications for using real time monitoring systems for recording and transmitting all data to the control center are presented with emphasis given on savings in both

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labor and time and need for the discovery of warning signs enabling raising earlier the alarm of possible failure or incident and the early taking of preventive measures. In this article it is argued that, in spite of the large investment of installing and running cost of comprehensive dam safety monitoring systems in tailings dams, such costs are justified as they form only a small percentage of the total investment in the tailings facilities projects, and may save huge costs if failure does happen. Such systems may be considered as an additional insurance against such events.

Keywords: tailings dam, safety monitoring, environment. ecosystem, visual inspections, instrumentation, deformation, seepage, phreatic surface, pore water pressure, total earth pressure, settlement, inclinometer.

1 Introduction

In the reported 18,000 tailings dams around the world, the failure rate of such facilities in the past 100 years is estimated at 1.2%. The failure rate of the traditional water storage dam is 0.01% [1]. The likelihood of Tailings dams' failures and accidents is more than 10 times than other conventional water retaining dams. Operating a tailings dam involves risks that need to be identified, quantified and mitigated. On average, three of the world's 3,500 tailings dams fail every year [2].

Statistics from available data shows that failures due to unusual rain increased from 25% pre2000 to 40% post 2000 due to the changes in climatic conditions, particularly at mine sites close to the seas and/or located in equatorial regions that have received high precipitations. The failures due to poor management accounted for 10% and 30%, respectively, for the same two time periods.

A comparison of the two time groups reveals that failure in dams of up to 15 m height increased from 28% to 30% whereas failure in dams with heights between 15 m to 30 m increased from 21% to 60%. The increase in failure of dam heights may be attributed to the combined effect of rapid dyke construction along with poor monitoring [3].

Rico et al., (2008) points out that tailings dams are more prone to fail than water storage dams due to some unique characteristics, including: (i) embankments built with residual material from mining activities, (ii) raising of the dam in multi-stages in order to increase storage capacity, (iii) lack of regulations and design criteria; (iv) continuous necessity of monitoring in order to assure the dam stability; and (v) high cost of maintenance after the mine closure [4].

Keeping the tailings pond safe and stable is the most challenging task in the entire mining process. The following points are the reasons why tailings dams are more susceptible to damage than other types of water storage structures: (i) embankments constructed with soil, coarse waste, and residual materials from the mining operations; (ii) the number of wastewater increases as the height of the tailings dam increases; (iii) lack of reasonable regulations on design standards; and (iv) the cost of monitoring the tailings dam is high during mine operation and closure [5].

Tailings' dams failure can results from several mechanisms. These mechanisms include seepage, foundation failure, overtopping, and earthquake. Considering the serious damages caused by tailings dam breakage, it is important to understand the main reasons and mechanisms for their failure, and adopt all possible means of enhancing their safety.

Major causes of Tailings' dams instability can result from: (i) seepage and internal erosion, (ii) Poor foundation conditions, (iii) overtopping), (iv) an earthquake effect causing static and seismic instability, and (v) other reasons such as mine subsidence, structural, external erosion.

2. Need for Monitoring and safety Reviews for Tailings Dams

The integrity and safety of a mine tailings dam is dependent on conservative design, diligent operation, and continuous, cost-effective monitoring. Geotechnical designs can be improved by increasing resistance factors; however, even dams with conservative characteristics may experience unexpected failure due to the presence of unidentified geologic features, abnormal weather patterns, seismic shock, and the inherently variable quality of a construction project that spans multiple decades and technological paradigm shifts. Diligent, well trained operators and a reliable monitoring system are therefore critical for failure prevention and loss reduction in the face of known and unknown unknowns. To this end, an adequate monitoring system should highlight the problematic behaviors that could lead to tailings dam failure with advanced warning. Experts in the field remind us that: "For any engineer to judge a dam stable for the long-term simply because it has been apparently stable for that long period of time is, without any other substantiation, a potentially catastrophic error in judgment" [6].

If any conventional storage dam during its operation is compared with the same hazard potential tailings dam located in a remote area during the closure phase, then very few dam safety guidelines, if any, differentiate in a meaningful way between such dams when stating dam safety program requirements. And yet, these requirements would most likely be significantly different. For instance, the conventional dam could be seen every day with surveillance walk-overs by site personnel or carried out on a weekly basis, while the tailings dam may be seen once a year only. Or, the conventional dam would be operated for some economic advantage such as flood control or power generation, while the tailings dam would not. Or, an emergency preparedness and response plans as well as a warning system could exist and be readily implemented for the conventional dam, while for the tailings dam such plans would not exist or, as a minimum, have different meaning and certainly different response protocols. Monitoring of contaminated groundwater would not be relevant to safety evaluations of the conventional dam, but may very well are required for tailings' dams. The list of such differences can be extended well beyond these sample examples.

The frequency and the extent of dam inspections and safety reviews must account for the practicality of the implementation of dam safety program. In many cases the extent and, possibly, the frequency of dam safety inspections would have to be more intensive in the case of a tailings dam during closure phase, ‘compensated’ by a lower frequency of dam safety reviews.

The frequencies of dam safety surveillance by site personnel, inspections and reviews recommended in dam safety guidelines often seem rather arbitrary, or customary at best. Yet, having a rationale for such frequencies may be highly desirable where the associated costs are high; like in the case of a remotely located closed tailings facility. At least in some respects such a rationale can be developed. As an example, one of the factors that might support specifying the frequency of dam safety reviews is the anticipated “possible” change in meteorologic or seismic database.

For instance, on a project in Canadian Arctic it was shown in 1995 that there was a definite cooling trend while, after extending the database by 7 years, it was concluded in about 2000 that a clear warming trend has been occurring at the site. As another example, on a project in British Colombia Province hydrologic designs had to be revised in 1998 after the original database was expanded by additional 7 years’ precipitation records. Alongside some other factors, such time periods may guide the frequency and, possibly, the extent of safety reviews for tailings dam during the closure phase. Obviously, the importance of climate warming or precipitation data will vary from site to site.

In terms of seismicity, for example in Canada, there has been a continual upward revision to the National Building Code of Canada seismic hazard maps and seismic ratings every five to ten years, showing remarkable increases in peak ground acceleration for infrequent seismic events in almost all regions of Canada; the last update was made in 2015 [7]. While an operating conventional dam may see one or two such major revisions, a closed tailings dam will be subject to changes in meteorologic and seismic databases for any concept of the foreseeable future. Having a focused guideline would be most useful to set up a framework for deriving a site-specific safety program for each tailings dam during the raising phase and in in the closure phase [8] and [9].

Monitoring of the tailings dam site during and after the mine closure is essential to evaluate the performance and effectiveness of the closure measures. The nature and duration of post closure monitoring depends highly on the nature and size of the operation and may continue for decades after the mine closure. On the basis of the information collected during monitoring, decisions related to further actions on the closed/rehabilitated mine sites can be made. The objectives of monitoring aim at securing proper mine closure. Therefore, a process of gathering information routinely, systematically, and purposefully for use in decision making is needed. The different closure objectives i.e. attaining physical, chemical, and biological stability and also having control over socio-economical influences, need different monitoring parameters that depend on site specific conditions. Technologies to be used in monitoring must be decided site-specifically at each tailings dam site after a thorough evaluation of their suitability [10].

3. Failure Modes of Tailings Dams and Safety Monitoring Requirements

Possible failure modes in tailings dams have to be recognized in any dam safety study. Parameters leading to any of them have to be defined and proper safety monitoring procedures have to be exercised in order to eliminate possible dangers of any failure and their consequences.

Key failure modes have been identified by the U.S. Committee on Large Dams (USCOLD) and the number of failures attributed to each mode is summarized from 106 incidents in Figure (1) [11].

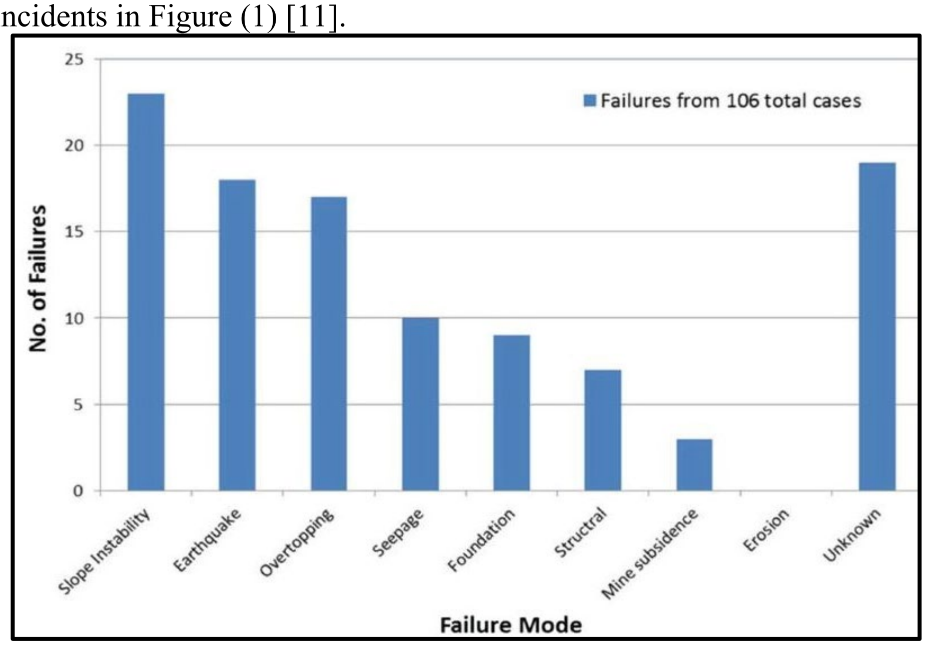


Figure 1: Failure modes of tailing dams and frequency distribution [11]

Slope instability failures, earthquake-induced failures, and failures owing to overtopping of the dam crest are identified as the top failure modes, while seepage, foundation, structural, and mine subsidence issues preceded fewer incidents. It should be noted at this point, however, that there is likely bias at work in these statistics as slope failures, earthquakes, and overtopping events are more easily identified in retrospect, and thus more readily attributed to the failure, than less conspicuous phenomenon like seepage, foundation instability, and internal structural deficiencies. Furthermore, it is recognized that this data represents the primary failure mode (i.e., that which resulted in a loss of containment) and that other minor modes and underlying mechanisms may have also contributed to ultimate failure. For example, the effect of elevated water held behind the tailings dam can contribute to a significant failure initiated by slope instability, overtopping, and seepage combined [12] and incipient slope failures very rarely occur without some deformation or surface expression.

An updated study carried out in 2019 shows different frequency distribution of failures and their reasons as highlighted in Figure (2). This may be due to more understanding of dam failures attributed to better monitoring, more use of instrumentation and wider reporting of failures cases of tailings dams during the last in the last 100 years.

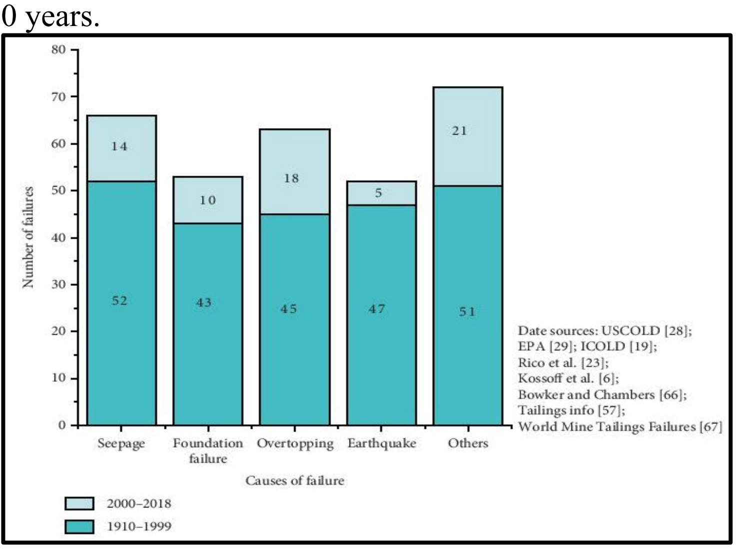


Figure 2: Failure modes of tailing dams and frequency distribution [13]

One scholar divided the causes of tailings dam breakage into 11 categories, namely, seepage or piping, foundation failure, overtopping, seismic liquefaction (earthquake), mine subsidence, unusual rain, snowmelt, structural, slope instability, maintenance, and unknown cause. But many accidents are the result of a combination of multiple causes, and this classification has some overlapping parts, such as unusual rain, snowmelt, and overtopping. According to the main cause of tailings dam failure, the accident percentage of seepage, foundation failure, overtopping, earthquake, and others, are respectively, 21.6%, 17.3%, 20.6%, 17.0%, and 23.5%. It can be seen from the collected data that extreme weather causes serious damage to the safety and stability of tailings dams. As human activities cause climate warming, the tailings dam failure ratio caused by extreme weather may gradually increase. It can be seen from the collected data that extreme weather causes serious damage to the safety and stability of tailings dams. As human activities cause climate warming, the tailings dam failure ratio caused by extreme weather causes serious damage to the safety and stability of tailings dams. As human activities cause climate warming, the tailings dam failure ratio caused by extreme weather may gradually increase [13].

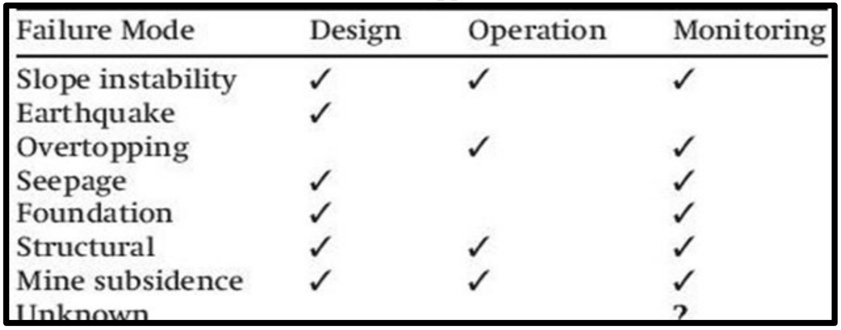
As failure investigations become more forensic in nature, our understanding of the interdependencies of the tailings dam system and the complexity of failure mechanisms become more apparent. The considerable number of unknown failure modes in Figure (1) and even in Figure (2) is indicative of a systemic, underdeveloped monitoring culture, a lack of established industry-wide best practices, and inadequate post-failure investigations. Unfortunately, a root cause analysis may indicate complacency or ignorance within the organization and its shareholders resulting in a lack of committed resources.

Tables (1,2) summarize the relationship between the possible failure modes and opportunities to reduce the risk of failure. Note that earthquakes and seismically induced failure is difficult to mitigate over any meaningful timescale and therefore earthquake-exposed sites must rely heavily on the design parameters and emergency response planning.

All known failure modes can be monitored and reasonably predicted given adequate information and an understanding of the expected behavior of the system. Table (1) enumerates several parameters of interest, changes in which are characteristic of behavioral changes in the dam system.

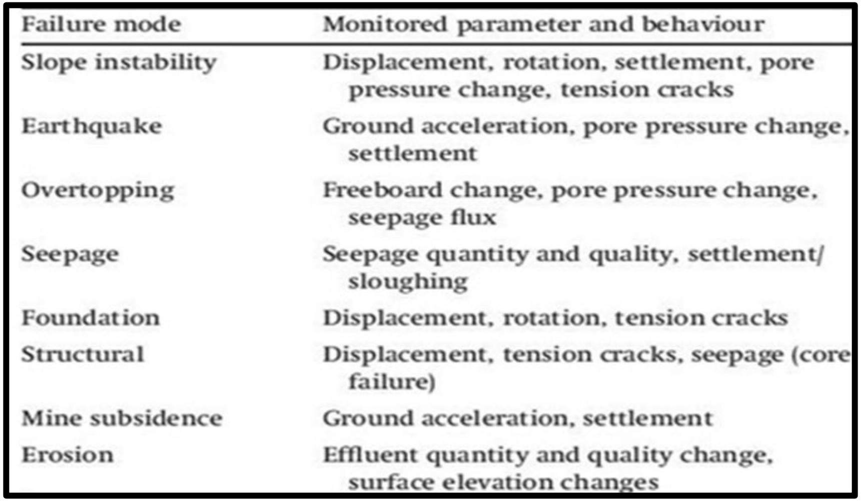
Table (2) enumerates several parameters of interest, changes in which are characteristic of behavioral changes in the dam system. The identification of undesirable behaviors is key to assessing the risk of failure [14].

Table 1: Failure modes and opportunities to reduce risk



Failure Mode	Design	Operation	Monitoring
Slope instability	✓	✓	✓
Earthquake	✓		
Overtopping		✓	✓
Seepage	✓		✓
Foundation	✓		✓
Structural	✓	✓	✓
Mine subsidence	✓	✓	✓
Unknown			?

Table 2: Failure modes and corresponding monitoring parameters



Failure mode	Monitored parameter and behaviour
Slope instability	Displacement, rotation, settlement, pore pressure change, tension cracks
Earthquake	Ground acceleration, pore pressure change, settlement
Overtopping	Freeboard change, pore pressure change, seepage flux
Seepage	Seepage quantity and quality, settlement/ sloughing
Foundation	Displacement, rotation, tension cracks
Structural	Displacement, tension cracks, seepage (core failure)
Mine subsidence	Ground acceleration, settlement
Erosion	Effluent quantity and quality change, surface elevation changes

In summary, the main mechanisms behind tailings dams' failures are clearly defined and they involve; seepage, foundation failure, overtopping, and earthquakes, as well as other mechanisms of the classic examples for tailings dam failures. Proper understanding of these, together with, sufficient supervision and good management of the operation of the facility help eliminate the causes of such failures and accidents.

It is not difficult to infer that each tailing dam failure involves engineering and human factors, and that these factors can be eliminated by; proper design and management procedures, stringent observations and measurements of dam behaviors and responses.

Finally, the availability today of wide range of modern measuring instruments and safety monitoring procedures can be considered as valuable assets in checking out the design, improving the performance of the required safety checks, which can come out with recommendations to safeguard the dam from possible adverse conditions.

4. Basic Principles of Tailings Dams Safety Program

4.1 Requirements

Any rational Tailings Dam safety program has to satisfy basic requirements if it is to give the anticipated results of avoiding incident and accident in such high investment projects. These requirements are explored in the following points;

i) **Continuity**

Surveillance is an essential element of the observational approach to design and construction that is so ideally suited, and so frequently applied, to tailings dams. Many tailings dam failure modes are preceded by warning signs that can be readily detected well in advance with a proper dam surveillance program. If the warning signs are promptly detected, and appropriately appreciated, time is available for the implementation of remedial measures. For many of these detectable failure modes, a good dam surveillance program, in concert with emergency preparedness and response plans, represents a highly cost-effective, proactive risk management tool relative to the options of a failure or an unnecessarily excessive and expensive degree of design conservatism. It does not however provide a means of safely stewarding a facility whose design and/or construction is flawed. Continuity of monitoring and surveillance of the facility is essential part of the successful and good management of tailings facilities including the dam, the pond and all other ancillaries. Need for rational safety monitoring program is established from the inception of the project and continuous long after its closure. Any meaningful safety monitoring program of tailings storage should be considered as an ongoing process covering the life cycle of the facility and it shall be carried out in all phases of its life.

Safety monitoring programs should be carried out by routine visual inspections, installing and regular reading of instruments and devices set for this particular objective, and by the appraisal of all the collected data by experienced engineers in safety reviews. Such safety reviews aim at the determination of the need for any further action to improve safety level of the tailings facility.

Therefore, any meaningful dam safety program has to be considered as a long chain of continuous activities, carried out in successive stages and extending over very long time which may last for decades.

ii) Phases of the Safety Program

Phases of safety program cover necessarily the complete life cycle of the facility from the planning stage of opening the mine, through the exploitation phase, and until post closure when all the wastes are deposited in the completed tailings dam. This last requirement is to ensure keeping the environment free from any detrimental effects of the deposits, and to detect, and remove, any danger to settlements, people life and their wellbeing. Actions needed in each of these phases may be summarized in the following lines.

a) At the preconstruction phase

At this phase, the method of raising the dam is determined and all possible environmental impacts on the surrounding areas have to be studied carefully. A complete file of past hydrologic and climatic conditions data at the site has to be accumulated including rainfall and snow fall information. These data are to be used as base line to compare for any future changes and possible consequence on the dam or on the pond. Projections of such data over the expected life cycle have to be estimated.

Changes on ambient ground water and surface water conditions have to be observed and kept to the absolute minimum. Therefore, at this preliminary stage ground water levels and water quality information have to be collected, and this has to continue during the service life of the project by the use of piezometers drilled for such purpose. These piezometers should be designed and installed as water sampling stations for taking water samples to be analyzed for any hazardous change in quality arising from increased content of harmful chemicals and heavy metals which can develop due to seepage water originating from the pond into the ground water table.

Surface water courses have to be monitored for possible discharge variations and possible flooding of the tailings dam and the tailings pond, and may require diversion of the courses of some of them. Collecting of such data at this stage is essential to carry out also Environmental Impact Study of the facility and to updated it in the future. This surveillance has to follow guidelines prepared by the designer which define the objectives, frequency and the means of such surveillance.

b) At the construction phase

The construction phase of any tailings dam covers the whole exploitation period of the mining facility which may extend for decades depending on the size of the facility and production rates. It is essential, therefore, to keep the tailings dam, the pond and all ancillary works under strict safety control if incidents or failure are to be prevented. This is done by safety surveillance through routine visual inspections, recording the readings of the installed instruments, and observing and documenting any anomaly in behavior which has to be reported immediately to the management for prompt action.

This procedure is the only mean to discover problems at an early stage and find solutions to ensure safety and avoid failure. the quick response to any threat that may appear during this phase by taking the appropriate remedial measures is of extreme importance.

Safety surveillance of tailings dams' is done in similar ways as for conventional dams and it follows guidelines drawn by the designer.

Use of instrumentation systems is normal in dam safety assurance programs of tailings dams during this phase, and it is similar in many ways to those used for conventional dams. Differences, however, do exist and these differences are detailed later on. More is said also on the types of measurements and the factors contributing to tailings dams' safety.

c) **At Closure and Rehabilitation Phase**

Immediately after closing down the mining facility, the rehabilitation phase begins and the question of the required monitoring of the completed tailings dam has to be addressed.

Depending on the particular details of the individual dam, various works may be carried out, such as the protection of the surfaces from water and wind erosion by covering with clean mine pit overburden or other available earth or rock materials. This may be supplemented by humus soil, sown with suitable crops. If toxicity is high, the crops may not be suitable for consumption, but if it is within admissible limits or reduced to such limits after a certain period, then it may be used for agricultural purposes. Slopes might be covered with earth grasses and afforested with rapidly growing trees and bushes, although care should be taken to ensure that surface runoff is intercepted by ditches and prevented from entering the tailings ponds. The tailings dam should be protected against extreme floods, and measures taken for complete interception and recycling of seepage water.

The monitoring usually required during the initial stages of rehabilitation includes measuring the seepage water discharge for its solids content, chemical composition and pH values, the position of the phreatic surface to confirm that it is safe within the impoundment and clear from the downstream slope of the dam.

Pore pressure conditions within the impoundment can be used to indicate consolidation, and should show up the effect of the drains, giving a clear indication of their effectiveness. Hydraulic measurements will indicate the effectiveness of the diversion and spillway facilities that have been provided, and settlements need to be checked to ensure that the hydrologic arrangements are not being nullified by the development of adverse gradients. The measurements and the observations should be carried out in the same way as the last years of operation. The same parameters are measured, i.e. those related to the stability of the dam, the erosion of the slopes and the protection of the environment. The same instruments are used so long as they continue to give reliable results, after the long period of operation.

The frequency of observations and measurements usually becomes progressively less during the rehabilitation phase and during the following long term period, depending on the success of the stabilization of the tailings dam and the reduction of dangers to environment.

4.2 Objectives of the Safety Program

Facility inspections should be performed by the competent authorities at all phases of the its life cycle. The objectives and their realization have been outlined already in the previous text but a reiteration may serve to emphasize the steps required for their accomplishment and the importance of eliminating any risk to the environment or human health.

One additional and important objective is to verify in particular if the tailings is managed in accordance with the applicable legal and regulatory standards, as well as with the approved operation manual and waste management plan. The objectives are, therefore;

(i) verification that the chosen location for the waste facility is going well with its performance and agrees with the assumed factors affecting its design and its construction,

(ii) verification that the physical stability of the waste facility is ensured and that pollution or contamination of soil, air, surface water or groundwater are prevented,

(iii) verification of the adequacy of the regular monitoring of emission and emission measurements,

(iv) verification if failures were properly reported and proper corrective action was taken,

(v) verification that the physical stability of the waste facility is ensured,

(vi) verification of the rehabilitation process is done well, including its proper documentation, and finally,

(vii) If the management of the tailings facility does not follow the operation manual and/ or waste management plan; whereby in such case, the governing authority should urge the operator to introduce corrective actions within a specified period, and if this is not done, to withdraw back the operation permit, [15].

(viii)

4.3 Procedures of Carrying Out the Safety Monitoring Program

The safety monitoring program is normally carried out in well-established and defined routines in important tailings facilities. It is done through; visual inspections by the facility engineers and preferably with participation from the designer staff, and by the periodic reading, recording and analyzing of instruments networks, and performing various measurements and tests in which the designer should also take part, and finally appraisal of the dam safety by the designer in safety reviews based on the accumulated data in the past years. If a critical condition appears at any time during this period, then this justifies carrying out an immediate safety review by the designer, and recommended actions are to be implemented to restore the required level of safety; this is explained in the following;

i) **Visual Inspections**

Visual inspections normally aim at looking for, and discovering of, at an early stage, cracks, evidence of any leakage, saturated areas and wet spots, appearance of springs, piping, erosion, sinkholes, slumping, frost heave, deviations in crest alignment, bulging or changes in slopes and depression of berms. Moreover, the drainage system has to be checked for any possible obstructions or damage by potential chemical reactions or bacterial growth. Sources of seepage, if any, have to be located and the rate and quality of water must be checked, while seepage volume is measured and related to the progress in height of the rising tailings pond level. All ancillary structures have to be checked for proper operation.

The frequency of visual inspections has to be decided by the designer at the start and may be changed by him during the service life, but an annual inspection is the normal practice and it shall cover all parts of the storage facilities; the dam, the pond and all ancillary works. If the tailings dam has been subjected to recent earthquake, then all after effects must be immediately noted and investigated and all damages recorded and appraised immediately not waiting for the routine inspection after finding out the magnitude of the event and its epicenter. Every inspection shall be proceeded by appraisal of the previous inspection report in order to check if previous recommendations have been performed, and if not why. The inspection has to follow a prepared and updated checklist including all items to be seen or measured. Findings of the visual inspection have to be compiled in a report but if any anomaly is observed then this has to be reported immediately. The inspection report has to include description of the methods used for measurements, summary of findings and recommendations for further actions and has to be supported by drawings and photographic documentations.

ii) **Use of Monitoring Instrumentation**

Performance monitoring by the use of instruments is an important way to determine if all structures are behaving as anticipated by the design. Doing this includes installation and use of such instruments as to observe the development of pore water pressure in the dam body, variation of phreatic surface level within it, settlement, displacements and deformations in the fill, and if the dam is located in seismic region then provisions must be made for measuring strong motion parameters at the site and dam’s response. While the readings of these devices is done on regular basis predetermined by the designer, their adequacy and proper functioning have to be checked also according to fixed schedules. Some dam Safety Regulation Authorities specify two years as a normal period for doing this.

iii) **Dam Safety Review Studies**

Regulating authorities normally require, from time to time dam safety review reports submitted to them by the managements as a condition for continuing the validity of the operation permit of the facility. The report of any such study must state in clear way the safety level of the tailings dam and pond, and that it does not pose any threat to third parties. Such report is compiled by the designer and shall be based on analyzing all the data and information made available to him from the previous safety inspections and instrumentations readings and measurements. Such reports take special weight after the occurrence of an accidents; in such a case, it must include full description of the accidents, its most probable reasons and its impact on the safety of operations. It shall also describe all the repair works, and any other works, that were performed to upgrade the safety level to the acceptable standards.

Recommendations to owners and operators who have specific responsibilities to apply safety management procedures to achieve safety improvement and risk reduction are to carry out the following actions which draw heavily on safety observation and monitoring.

i) Detailed site investigation by experienced geologists and geotechnical engineers to determine possible potential for failure, with in situ and laboratory testing to determine the properties of the foundation materials,
ii) Application of state of the art procedures for design,
iii) Expert construction supervision and inspection,
iv) Laboratory testing for "as built" conditions,
v) Routine monitoring,
vi) Safety evaluation for observed conditions including "as built" geometry, materials and shearing resistance. Observations and effects of piezometric conditions,
vii) Dam break studies,
viii) Contingency plan,
ix) Periodic safety audits. Regulatory Authorities should be more concerned about the safety of tailings dams that come under their jurisdiction and should require periodic reviews carried out by appointed specialists. In some countries approval had to be obtained for specific stages of construction, causing the stability, general condition and safety to be automatically checked from time to time [16].

4.4 Comparison of Basic Requirements for Tailings Dams vs. Conventional Dams

Tailings dam surveillance must serve three vital functions of:

i) Confirm that the dam is safe, and is performing in accordance with design criteria and objectives.
ii) Confirm design assumptions, and adjust the ongoing design and construction of the dam as necessary based on observations less favorable than design assumptions, or optimize design and construction based on

observations more favorable than design assumptions.

ii) Clear identification of failure modes capable of detection by the surveillance program, and those not capable of detection, so as not to lead to complacency with regards to the latter.

The continuity of works in tailings dams over the whole life of the project and the nature of the purpose and methods of construction make these dams different from conventional dams in so many aspects Although the need for preserving high safety levels is common for both, surveillance requirements may differ. Table (3) shows a comparison between the two types requirements.

Table3: Tailings dams vs. conventional dams in terms of surveillance requirements

Conventional dams	Tailings Dams	Implications for tailings dam surveillance program
Construction occurs in a single stage, followed by operation under steady state conditions.	Construction occurs on a regular, near-continuous basis. Steady state is not achieved until closure	The conditions of the dam and the loadings to which the dam is subjected, and therefore surveillance requirements, are constantly changing. There is no opportunity to "relax".
Constructed and operated by organizations experienced and focused on dam operation and surveillance.	Constructed and operated by organizations whose focus and expertise is in extracting wealth from the ground, not in building dams.	Operations personnel require training, support, and commitment from corporate to mine management level.
Typically owned by state or large utility with substantial financial resources to apply to dam safety.	Economics of mining limit the resources that can be applied in construction and operation of the dam.	Tailings dam owners must resist the inevitable temptation to cut back on surveillance program in favor of devoting more resources to profit centers. Resources applied to surveillance must be independent of cyclical business environments.
Viewed as a significant asset (generates revenue) by the owner and, typically, by the public which derives direct, tangible benefit from the dam.	A cost center for the mining operation, a "necessary evil", and understandably not the focus of the operation. Definitely not viewed as an asset by the public	As above, tailings dam owners must resist the obvious and understandable temptation to reduce resources available for dam surveillance. Must not just practice due diligence, but be seen to be doing so to reassure regulators and the public.
Owners generally have inhouse dam engineering expertise, including personnel whose sole function is surveillance.	Typically, no in-house dam engineering expertise, reliance on consultants. Dam surveillance is not a career path for miners.	Disconnect exists between those carrying out the surveillance program (operators) and the dam engineering expertise required to evaluate the results of the program (consultants). Continuity of advice, and clear communications essential.

Have a finite operating life, do not have to last "forever".	Have a closure phase as well as operational phase, have to last "forever".	Most tailings dams will require some degree of surveillance "forever", particularly so for the increasing number of water retaining tailings dams being used to keep sulfide tailings submerged.

In spite of the different goals and methods of construction, tailings dam surveillance requires all activities necessary for achieving safety and continuity of service similar to conventional dam, and should include all of the ingredients as in conventional dams i.e. determination of the data that needs to be gathered to confirm satisfactory performance, design of a surveillance program that will obtain the required data, with a suitable level of redundancy (repeatability), training of those personnel responsible for collection of the data as to how to obtain and manage the data and to provide awareness as to warning signs, provision of the installations and resources required in the form of instrumentation, time, personnel, access to permit responsible personnel to carry out the program, maintenance of the installations such as but not limited to piezometers, inclinometers, seepage weirs required to carry out the program and establishing a level of "respect" for installations within all on-site personnel to avoid, for example, damage to critical installations by vehicle traffic.

It is most important in tailings dams' operations to present at all times data in the form of plot of piezometer level vs. time, and water levels including trigger levels to allow for quick and effective interpretation and response. Prior definition of warning signs and response plans are to be enacted in the event that warning signs are noted. Due to the need for quick response in emergencies clear and documented lines of communication and responsibility in terms of data gathering, plotting, reporting, and action are needed [17].

In tailings dams, maintenance of reasonable safety standard is the joint work of the owner, operators, designers, safety reviewers, and the regulators, but contrary to conventional dams it requires much deeper way of public participation. It requires the recognition of the vital importance of the mining industry to modern society and to the economies of developing countries, and that elimination of mining must eventually mean a return to a Paleolithic lifestyle, it requires also continual of responsible stewardship of the environment by this essential industry and the acknowledgment that the vast majority of mining industry operators and operations deserve praise for their efforts [18].

5. Tailings Dams Failure and the Need for Instrumentation, Measurements and Observations

Understanding of the failure modes of tailings dams is very important step in assessing their safety, and it requires prior understanding of what can go wrong and what input data are required. Such data may only be obtained from visual observations, instruments recording, measurements and laboratory testing which all form a complete safety monitoring program. In the following a brief description of failure modes is presented, so that the required measurements and types of instruments needed are to be indicated later on;

i) Rotational failure

ii) Rotational failures can range from local sloughing of tailings at random areas along the face of an embankment to massive circular arc slides extending over the entire structure. In general, for a stable slope, the shear strength resisting movement along a potential failure surface exceeds the shear stress tending to induce movement. Instability occurs when the shear stress on the failure surface equals the shear strength. Specifically, causes of rotational failure may include changes in the water table, changes in the permeability of the foundation materials, disturbances to the embankment caused by vibration or impact loading, settlement of the foundation materials, etc.

iii) Foundation failures

iv) Foundation failures are not uncommon among earthfill structures. Where a weak layer of soil or rock exists at shallow depth in the foundation below the structure, movement along a failure plane will occur if the earthfill loading produces stresses in excess of the shear strength of the soil in the weak layer.

v) Overtopping

vi) One of the most common causes of failure is overtopping by flood waters. Overtopping typically results when the volume of run-on entering an impoundment, from improper diversion of surface water flows or excessive stormwater flow, exceeds the capacity of the impoundment. Because tailings embankments are constructed of highly erodible materials, the friction caused by rapid flow over an unprotected embankment crest may quickly erode a gully in the fill material, allowing sustained release to occur. Additionally, a rapid increase in pore pressure associated with large stormwater inflow may result in the liquefaction of unconsolidated impounded sands and slimes. Sustained high flow over the crest of an embankment can thus result in a major failure of the overall impoundment within minutes.

vii) Erosion

viii) In areas of heavy rainfall, some form of protection against erosion is usually required. Tailings embankments may be susceptible to erosion failure in two major areas, embankment abutments and the embankment face. Erosion along the contact line between the embankment and the abutment may result from stormwater flow that concentrates there. Typically, this type of failure is preventable with proper stormwater diversion methods and so results from faulty design or maintenance. Erosion of embankment faces may result from rupture in tailings pipe lines installed on the embankment crest. Again, maintenance and alternate siting of tailings lines may prevent this type of failure.

ix) Piping

x) Piping refers to subsurface erosion along a seepage pathway within or beneath an embankment which results in the formation of a low-pressure conduit allowing concentrated flow. Piping may result also from seepage exiting the face of an embankment with sufficient velocity to erode the embankment face. The resulting void space promotes progressive erosion extending upstream toward the source of the seepage. In the worst case, the seepage may result in the creation of a direct channel from the tailings pond to the dam face Excessive piping may result in local or general failure of the embankment or the embankment foundation.

xi) Liquefaction

xii) Liquefaction is one of the most common failure modes of cross-valley dams. Because tailings deposits typically comprise unconsolidated, saturated deposits of similarly-sized grains, they are susceptible to temporary suspension in water Liquefied tailings may behave like a viscous fluid, such that they may pass through narrow openings and flow considerable distances. Accordingly, even small dam failures may result in substantial releases of tailings flow.

6. Safety Parameters and Use of Instrumentation

In the followings the main parameter contributing to the safety of tailings dams are presented with description of the methods, ways and means used in safety monitoring programs;

i) Dam saturation line monitoring:

One common practice to monitor the saturation line or phreatic surface level variation in the embankment, and therefore conditions of pore pressure, is to use open pipe piezometers. The pore water pressure regime in a tailings dam, or any other earthfill dam for that matter, has a significant influence on the stability of the dam wall. Single standpipe piezometers at various cross sections around a tailings dam, is commonly used to monitor the pore pressure in the area close to the outer wall. Typically, piezometers data is recorded on a monthly basis and used in stability analyses. Once a base case analysis has been carried out, further analyses are usually only carried out once significant changes occur in the piezometric values are recorded. Sometimes, other piezometers in the form of pore pressure measuring cells are used. Two types of this sort are available; the first is the diaphragm type, where a diaphragm is placed between the transducer and the pore water. This is a pneumatic vibrating wire, or electrical resistant strain gauge; the second type is a standpipe without diaphragm and uses twin tubed hydraulic construction which is known in literature as the Casagrande type piezometer.

The construction of standpipe piezometer consists of porous filter element at the tip so that the instrument responds to subsurface water pressure around the filter element and not the fluctuations at other elevations. The water level in the stand pipe stabilizes at the piezometric elevation and is determined by sounding with a probe which is normally electric dip meter.

In such case, the water level recorded in single standpipe piezometer cannot measure the fluctuation of the phreatic surface and therefore such variation can only be obtained from multiple number of such piezometers installed in the same cross section and having their tips at different elevations [23].

Following the measurements of phreatic surface using piezometers helps to indicate the safety stability status of the dam and leads to executing remedial measures to improve the safety conditions.

According to Swedish dam safety guidelines, the factor of safety in terms of slope stability of tailings dams should be at least 1.5 under normal conditions. In order to prevent the factor of safety to be lower than 1.5, strengthening actions are needed for the dam. In actual study case, rockfill embankment was added at the downstream toe, and in order to optimize the amount of rockfill, the rockfill support plan was followed by site measurements of the phreatic surface elevation by the installed piezometers and calculating the stability conditions of the dam. Figure (3) indicates the measured phreatic water surface before the addition of the rockfill berm and the location of slip surface in the dam. Figure (4) on the other hand shows the actual conditions after construction of the embankment.

Since the safety of the dam is maintained even though excess pore water pressures exist, a proposed alert level is therefore the limit where hydrostatic pore water pressure is exceeded. The hydrostatic pressure is here based on a phreatic line at the tailings surface. If the pore water pressure measured at site is lower than this alert level, the measurement is said to be in the green stage. The factor of safety is higher than 1.5 in the green stage, while the yellow stage the stability is ensured even with some excess pore pressure, but stability has to be observed with caution. Passing this stage into the red stage would be intolerable and has to be prevented. which is schematically shown in Figure (5)[24].

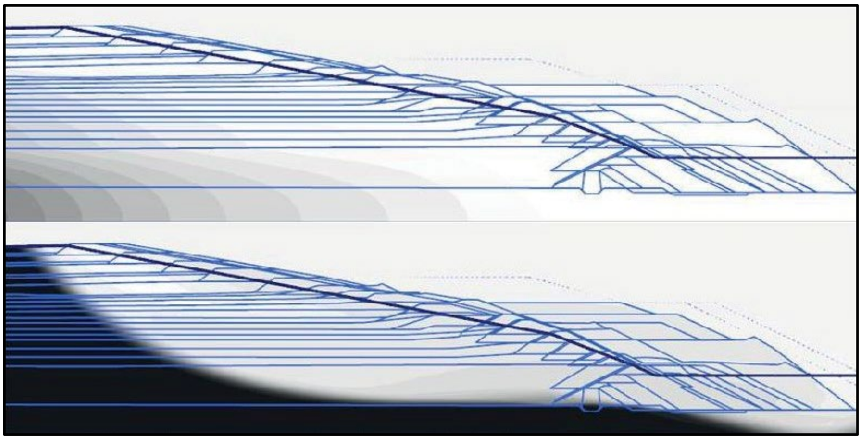


Figure 3: Before embankment construction 2015. Upper: Simulated excess pore pressure. *Lower*: Most probable slip Circle (Factor of Safety 1.58) [24]

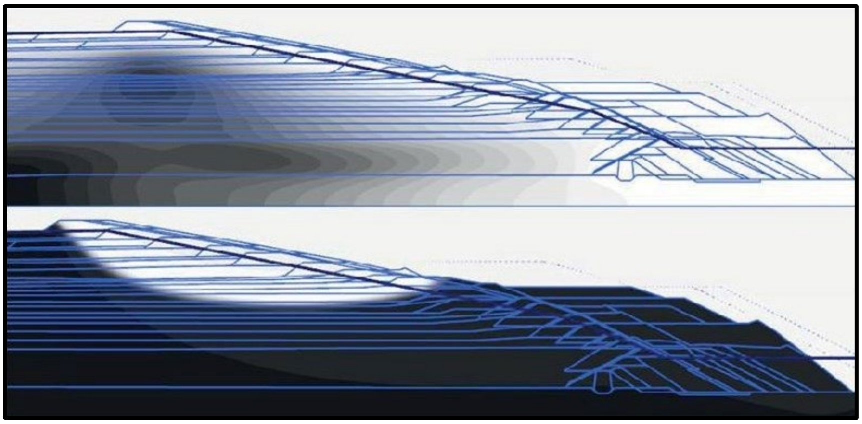


Figure 4: After embankment construction 2015. Upper: Simulated excess water pressure. *Lower:* probable slip surface (factor of safety 1.49) [24]

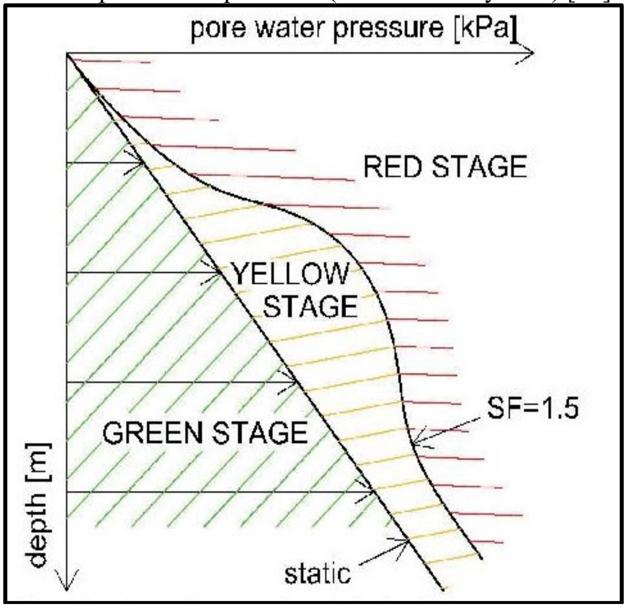


Figure 5: Schematic presentation of alert levels for pore water pressure (The three alert stages are, green, yellow and red) [24]

ii) Water Levels Monitoring:

One of the most critical parameters for the safe operation of tailing dam is water level control, not only of water level in the pond but also of surface water and the water table underneath the dam. Monitoring the level of surface water can give an early indication if there are sudden rising levels due to floods, rain or ice melt. A radar level transmitter is a great fit for this type of application. When it comes to the water table, it is done by installing hydrostatic level sensor in piezometers.

With regard to the maximum safe water level in the pond, its determination during the design stage is important to avoid overtopping of the tailings dam which can lead to its failure and release of dangerous waste.

Normally, the minimum safe freeboard is determined by national/local legislation and/ or company policies. It is essential in the design stage that water balance calculations consider average and extreme conditions to prevent loss of freeboard during operation which may result in overtopping.

Freeboard management, and pond water level monitoring are critical factors for the control water of both conventional and tailings storage impoundments [25]. To promote good drainage of the embankments, the supernatant pond has to be as far away from the embankments as possible and maintain a high freeboard and wide beach. Ideally the pond should be of a small area, to reduce seepage, and be at a maximum depth at the decant facility to improve reclaim water clarity. Figure (6) shows examples of loss of freeboard and high freeboard.

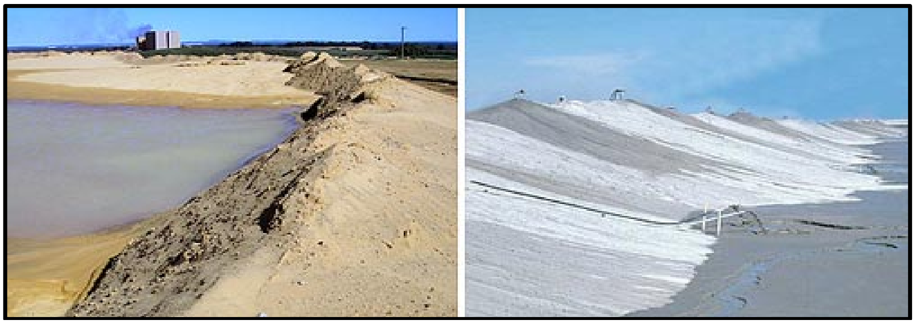


Figure 6: Loss of freeboard (left), and high freeboard (right).
(© Jon Engels, Right picture courtesy of Anglo American) [25]

The diagrams shown in the following Figure (7) explain the definitions related to freeboard in possible conditions of the pond.

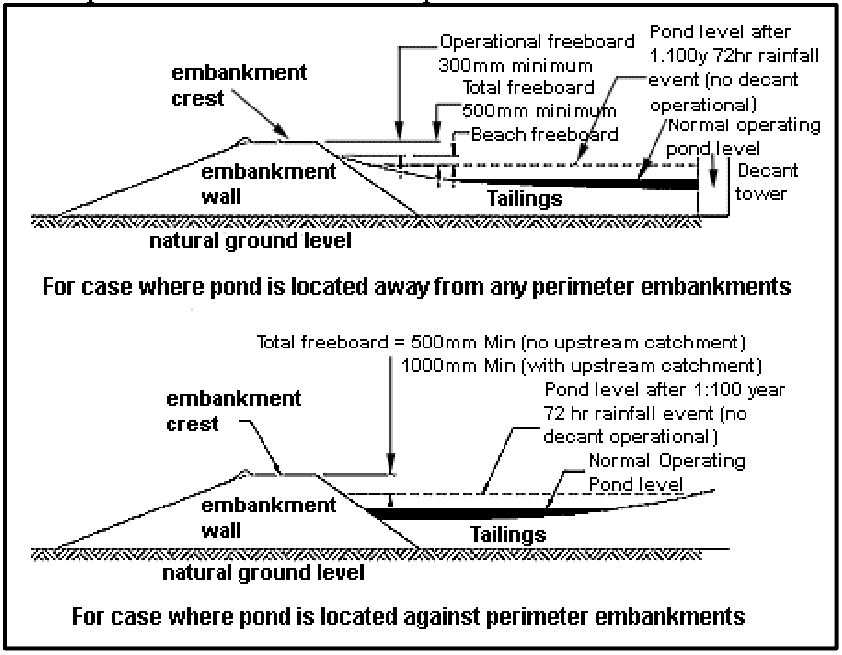

embankment crest
Operational freeboard 300mm minimum
Total freeboard 500mm minimum
Beach freeboard
Pond level after 1:100y 72hr rainfall event (no decant operational)
Normal operating pond level
Decant tower
embankment wall
Tailings
natural ground level
For case where pond is located away from any perimeter embankments
Total freeboard = 500mm Min (no upstream catchment)
1000mm Min (with upstream catchment)
embankment crest
Pond level after 1:100 year 72 hr rainfall event (no decant operational)
Normal Operating Pond level
embankment wall
Tailings
natural ground level
For case where pond is located against perimeter embankments

Figure 7: Freeboard of a tailings facility explained
(Adapted from (DME 1999) (© Jon Engels) [25]

Freeboard loss can lead to overtopping which leads in its turn to the rapid failure of the tailings dam, such loss may result from damaged diversion spillway or blockage or, or misjudgment of the actual water level and wrong design criterial. As shown in Figure (8), checking of the spillway conditions and has to be done, tailing pond water level has to be monitored continuously by installing water level sensors and sounding devices. Real time measurement, recording and telemetering on continuous basis instrumentations are available and for large facilities their high cost is justified [26].

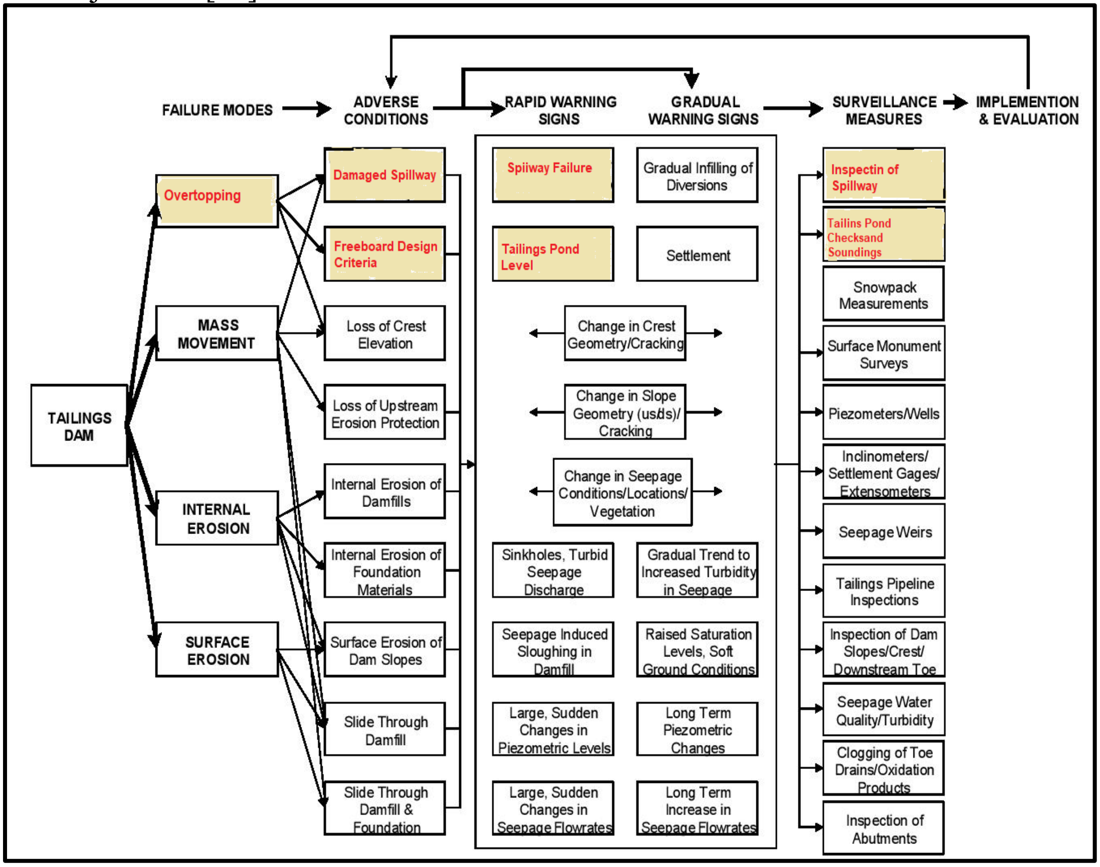

FAILURE MODES
ADVERSE CONDITIONS
RAPID WARNING SIGNS
GRADUAL WARNING SIGNS
SURVEILLANCE MEASURES
IMPLEMENTION & EVALUATION
TAILINGS DAM
Overtopping
MASS MOVEMENT
INTERNAL EROSION
SURFACE EROSION
Damaged Spillway
Freeboard Design Criteria
Loss of Crest Elevation
Loss of Upstream Erosion Protection
Internal Erosion of Damfills
Internal Erosion of Foundation Materials
Surface Erosion of Dam Slopes
Slide Through Damfill
Slide Through Damfill & Foundation
Spilway Failure
Gradual Infilling of Diversions
Tailings Pond Level
Settlement
Change in Crest Geometry/Cracking
Change in Slope Geometry (us/ds)/ Cracking
Change in Seepage Conditions/Locations/ Vegetation
Sinkholes, Turbid Seepage Discharge
Gradual Trend to Increased Turbidity in Seepage
Seepage Induced Sloughing in Damfill
Raised Saturation Levels, Soft Ground Conditions
Large, Sudden Changes in Piezometric Levels
Long Term Piezometric Changes
Large, Sudden Changes in Seepage Flowrates
Long Term Increase in Seepage Flowrates
Inspectin of Spillway
Tailins Pond Checksand Soundings
Snowpack Measurements
Surface Monument Surveys
Piezometers/Wells
Inclinometers/ Settlement Gages/ Extensometers
Seepage Weirs
Tailings Pipeline Inspections
Inspection of Dam Slopes/Crest/ Downstream Toe
Seepage Water Quality/Turbidity
Clogging of Toe Drains/Oxidation Products
Inspection of Abutments

Figure 8: Overtopping mode failure path, explained, Inspections of spillway and tailing pond plus checking and monitoring water level indicated (All in yellow color) [26], (Figure modified by the writer)

Modern technologies are available today for monitoring precisely tailings pond level by installing a complete system assembly on a barge that floats on the pond and connected to the embankment by a bridge.

An actual example provided with real-time monitoring capabilities and powered by solar energy is used now by a Canadian oil sands mining company and it is illustrated in Figure (9).

In this system, cellular data communications is used to provide connectivity to site (RTU) and to a (SatSCADA) server. Differential (GPS) transceiver (DGPS) and Radar Level Sensors (RLS) are used to achieve required accuracy and to provide the correct (GPS) level of the Barge.

A Radar Level Sensor (RLS) is connected to data logger which is used to measure the distance to the water line. The (RLS) distance combined with the (GPS) reference level is used to calculate the exact water level. All data is sent periodically back up to the (SatSCADA) server for archival, trending and display. Users access the data is done via (SatSCADA) web client which can provide access through to laptop, desktop, tablets, and smartphones, the complete instrumentation set up is shown in Figure (10) [27].

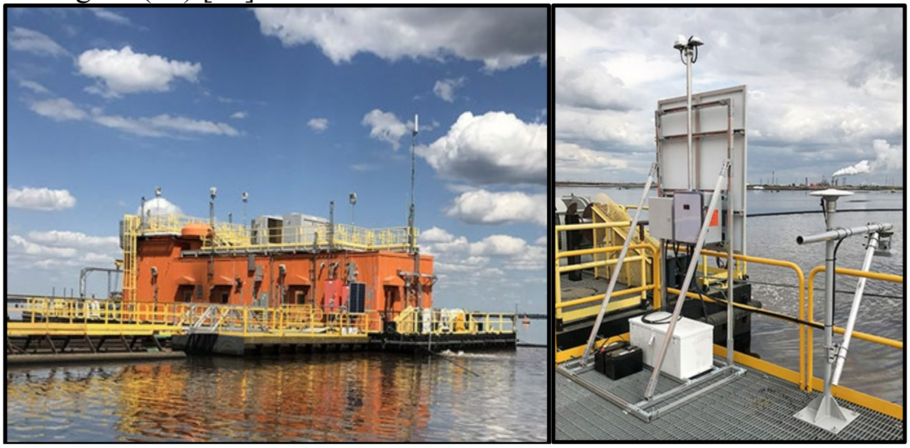


Figure 9: (Left), view of the barge. (Right), The sensor and transmission arrangement

iii) Seepage Outflow Discharge Measurements

All seepage water should be collected, and the discharge measured by weirs of V- notch, rectangular or trapezoidal type. Calibrated containers might be used to measure low discharges.

Seepage discharge is the most important factor which has to be measured in respect of dam stability and it can provide evidence of any serious failure in tailings dams. It is indicative of internal erosion in the dam body with grave consequences if all the necessary measures are not taken in due time. This measurement should be automated and the readings recorded without interruption in the control room. It is appropriate to register separately the discharge coming out from each drain and place, so as to be able to determine more easily the location of any increase of seepage and the possible occurrence of erosion.

It is essential for tailings dams to be well drained, to keep the phreatic surface as far away as possible from the downstream slope, to reduce pore pressure and to decrease the danger of liquefaction. The drainage augments the seepage discharge which is much greater than in earthfill dams. However, it is not dangerous if there are properly functioning filters and drains, and if the seepage quantity is collected and turned back to the tailings pond or the processing plant.

Observational data from several operating tailings dams indicate that flow nets may be used to estimate seepage flows and to determine the location of the phreatic surface through the beach and tailings dam with sufficient accuracy for most design purposes. The greatest unknown is the effective permeability of the tailings beach and it is recommended that the maximum probable values for both the permeability and the permeability ratio (kmax /kmin) be used for all computations. This procedure will produce conservatively high values for both the seepage quantity and the location of the phreatic line.

Seepage control measures unquestionably add to the total costs of the tailings storage facilities. However, they greatly increase the overall safety of the tailings dams and when properly designed and constructed satisfy both the safety and pollution control requirements of the regulatory agencies. The additional costs associated with constructing adequate seepage control measures should be considered a necessary part of the cost of building and operating a mine in today's society [28]. Needless to say, that if this is coupled with good observation and monitoring procedure will enhance the safety conditions and avoid surprises due to unaccounted conditions

iv) Seepage and Drainage Water Quality Measurements

The environment can be affected by mining operations in numerous and reactive ways. The production of Acid Mine Drainage (AMD) and contaminated seepage water caused by oxidation of the sulfide minerals in the presence of water is probably the main threat to many ecosystems. AMD is recognized as a multi-factor pollutant; the main factor being acidity, toxicity, suspended solids and salinization.

One of the main goals of tailings water management is to ensure that water quality in surrounding bodies of water is not degraded by drainage and seepage water generated from tailings ponds. To that end, some manufacturers can offer both stationary and portable water sampling stations in full compliance with all relevant national and international standards. Such sampling and the following measurements can help in deciding on the best suited technology for treatment, size of treatment plants to be used before returning the collected affluent to the environment.

The chemical composition of tailings seepage and drainage water is important in determining any potential environmental impacts. Factors include waste characteristics such as mineralogy of the host rock and milling methods used to produce the tailings, and the interaction of the tailings seepage with the liner, if any, and the subsurface. Contaminant mobility can be increased by physical mining processes such as milling, whereby, a small grind results in increased surface area for leaching. Most mining companies manipulate pH and use chelating agents to extract minerals from the ore. These same processes can be applied to the fate and transport of contaminants in tailings. While many heavy metals are hydrophobic with strong adsorption tendencies for soil, the chemical reagents used in mining processes may be present in the tailings material. They are able to remove the metals, making them mobile in leachate or surface waters.

Contaminated water may be formed from downward migration of impoundment constituents or ground water movement through tailings. Most contaminant transport in ground water systems is from the advection i.e. fluid movement and mixing of contaminants. The geochemistry of the aquifer, physicochemical properties of the tailings and seepage will determine the buffering capacity of the soil, types of chemical reactions, whether in precipitation or neutralization, and the rate of adsorption and ion exchange.

A related problem is the production of acid by oxidation of thiosalts, which is a problem for some metal mines in eastern Canada. The bacterial culprit is thiobacillus thiooxidans. Thiosalts may be removed from the mill effluent by biological treatments [29].

In all the aforementioned details, a clear indication of the need for measurement of tailings affluent quality is recognized. This is considered as the first step towards their amelioration and removing the contamination threat they pause to the environment.

v) Inflow Water Control Measurements

Measuring what goes in the tailing ponds is the first step in responsible tailings management. Measuring the flow of the ore slurry is not enough. However, the solids content of the slurry can also be found and the relation between solids and water calculated.

By using an electromagnetic flowmeter, one can measure both parameters with just one instrument. When it comes to quality measurement, digital analytical sensors can tell which substances are entering the tailings. Such flowmeter mounted on delivery pipes to the tailing pond is shown in Figure (11) [30].

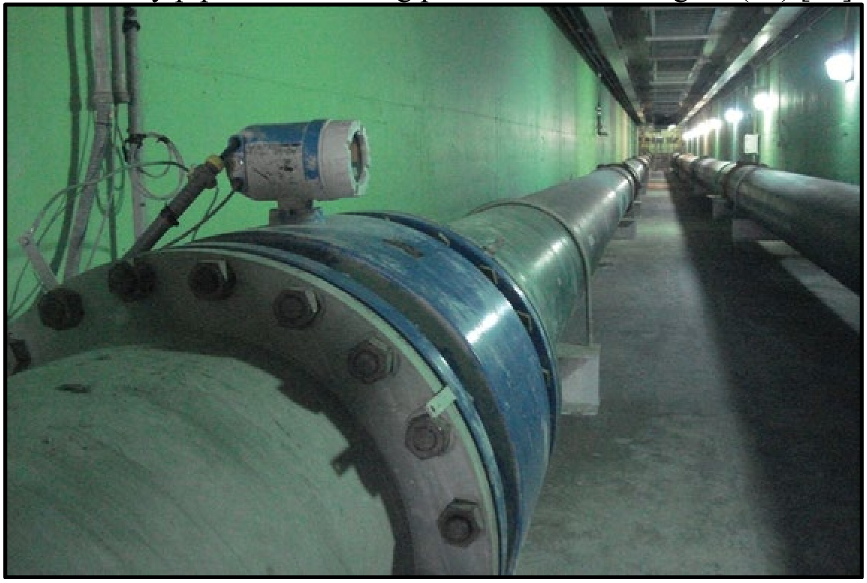


Figure 11: Endress and Houser type electromagnetic flow meter [30]

vi) Monitoring the Vertical Displacement of the Dam

Vertical movements in tailings deposits are large and are due to compaction from its own weight and to consolidation. The external movements are measured by means of a survey control pillars and geodetic datum points network. The control points or pillars are placed sufficiently outside the dam to avoid the influence of dam weight. The geodetic monuments are provided in sufficient numbers and are located along the crest of completed stages, berms of the downstream slope, along the crest of completed tailings dam and on the beach of completed dam.

Monitoring the vertical displacement of the dam can also be done by vibrating multi-variable testing meter, using a vibrating wire displacement sensor as a measuring head. The measuring head, through a fibre glass rod and anchor head, are connected according to design requirements at different depths in the borehole and the surrounding soil with its mobile sensor and can measure vertical displacements at different levels [31].

vii) **Horizontal displacement monitoring**

In tailings dams, the external horizontal movements are due mainly to non-uniform settlement of different parts and much less to pressures from retained water. They are measured at same geodetic positions used for surveying the vertical movements. More exact measurements can be performed when the points are located in one straight line and the movements are measured in relation to it. If the dam crest and berms are not in straight lines, less accurate trigonometric methods can be used. Methods in this case are the optical clinometers or the sophisticated laser instruments. Careful study of successive sets of stereoscopic aerial photographs can reveal horizontal movements that have occurred, and may be particularly used for checking a long dam.

Within the body of the dam horizontal movement can be detected by inclinometers. Inclinometer is installed in a pipe string. Layer and structure movement cause inclinometer displacement and force the inclinometer tube to deform from the initial position to a new location. Inclinometer sensor measures the rake and from its deviation from its original slope displacement can be obtained. Inclinometer tubes are embedded from the bottom of the deepest starting point of each segment and it reflects the displacement of soil at that level.

The horizontal movement of a deformable foundation can be measured with inclinometers or inverted pendulum, and may be used absolutely for the starter dam due to their high cost. Use of GIS is also justified for use in large tailings dams.

Large movements are not so dangerous if they are distributed over large areas and if their rate decreases with time, on the contrary, small movements can be dangerous if they are concentrated at one place and if their rate increases with time.

Generally speaking, high precision deformation measurement can be done using GPS precision positioning technology, compared with classical measurement, this cannot only meet precision requirement (within±1mm) of dam deformation monitoring, but also help to realize automation of monitoring work. For instance, in order to monitor the dam deformation, technicians select a GIS Continuously Operating Reference Stations (CORS) and a number of monitoring points in deformation area far from appropriate position of dam. In the base station and management center, the technicians install GPS receivers respectively which carry out continuous automatic observation. By using proper data transmission technology, monitoring data is real-time transferred to data processing center to carrying out treatment, analysis and prediction of observation data, Figure (12) [31].

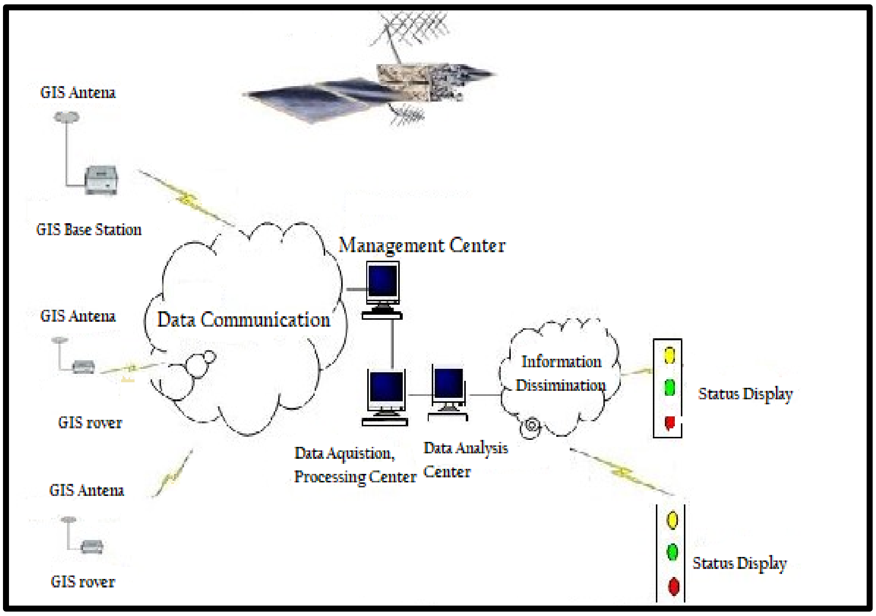


Figure 12: Structure of GIS System [31]

The precision of the horizontal displacement and vertical displacement, obtained by using precision broadcast ephemeris and 6～8 hours dual-frequency GPS observations, is better than 1.0 mm. Applied in building monitoring, GPS technology can not only be free from limitation under the intervisibility and climate conditions, which carry out all-weather monitoring in heavy fog and snowstorm conditions, but also can collect transmit and manage data, analysis deformation and automatically forecast. So, it can reach the purpose of remote online network real-time monitoring. Besides, GPS can still carry out dynamic deformation monitoring and export the result of real-time positioning according to10～20Hz frequency.

Baseline vectors, produced by GPS relative positioning, can obtain high precision geodetic altitude by survey adjustment. As the vital observation here is the relative settlement during vertical displacement monitoring. We could, without conversion, directly use geodetic altitude system, which will not only simplify calculating work, but also ensure precision of observation results [32].

viii) Differential Settlement of embankment

The differential settlement in the embankment can be measured with internal vertical movement measuring devices. Most suitable are PVC standpipes with settlement plates or rings made of stainless steel fitted over the pipes and an electromagnetic probe lowered in the standpipe, to measure the position of the plates. The plates are placed on the surface of the rising tailings, and spaced at certain vertical interval (most frequently about 5 meters). They are commonly 30- 40 cm across, and move together with the settling tailings, sliding over the pipes.

Since the tailings suffer large settlements, it is not appropriate to use telescoping pipes which are applied in earthfill dams, because settlement plates may get stuck at the splicing. It is preferable to use tightly and smoothly spliced pipes to allow the plates to slide unhindered over the pipes.

Settlement devices are usually used to obtain the following information:

a. The compaction of the deposited tailings and its variation with time at different heights of the dam.
b. The settlement of the foundation.The measurement of settlement and pore pressure gives information about the consolidation of the tailings at full scale and provides in- suite data, supplementing any laboratory consolidation tests that had been made to obtain information for design. One can determine from them the deformation modulus and pore pressures coefficient (ru) for every layer, as a function of load and time. These functions are input data to give more reliable calculation of the stress- strain state and consolidation of the dam.

ix) Total pressure measurement

The measurement of total pressure is used for three purposes;

a. to obtain the effective pressure, which is required for calculating the stability of the tailings dam. This is given by the total pressure, after subtracting from it the measured pore pressure.
b. The acting pressures on structures such as the decant towers or chutes, culverts etc. required in order to check design calculations and overall stability.
c. The total pressure in the horizontal and vertical directions to obtain the coefficient of total pressure and also Poisson's ratio, which can be assumed here as simple stress dependent.

For the total pressure measurement similar instruments as those used in conventional earthfill dams may be used also.

x) Monitoring Seismicity

Large tailings dams and particularly those located in seismic regions, require so- called "strong motion accelerographs" to register the stronger earthquakes above a certain given threshold, which can be varied at will, and it is usually 0.1g. As a minimum, two or three accelerographs should be used. One accelerograph which is the "main" or the "master", is placed on the bedrock, to first register the earthquake. It sends signals to a computer that controls the others, that are placed in shallow excavations at two different places in the upper part of the downstream slope. They are periodically moved during operation so as to be in the upper part of the slope, and after the tailings dam ceases to operate, they are placed on the beach in the tailings. They are used for determining the tailings damping response, required for calibrating the seismic numerical model and calculating the earthquake response of tailings dams.

Other than the dam, the mine open pit can also be monitored for micro-seismicity by accelerographs which are mounted in shallow holes on or near the top of the mine. An example is shown in Figure (13) of such instruments of SP2 type model used for monitoring micro seismicity of a mine open pit which is produced by Sure Wave Technology Ltd.

Such technology can be used to continuously monitor and detect micro-seismic events occurring within the slope walls of open cut mines and other surface material deposits such as tailings dams, giving advance warning of internal movement and the potential for collapse.

Each sensor can typically monitor micro-seismic activity up to a certain distance depending on the model used. The micro-seismic activity can be displayed on a 3D image and updated in real time. The micro-seismic events displayed are indications of stress build up within the side walls or dam structure with the potential to lead to a failure. In addition, water pore pressure variation can be mapped in real time to visualize what is happening inside the structure. For a dam, this may include pore water inside the tailings leading to movement of the base material in a liquefaction mode.

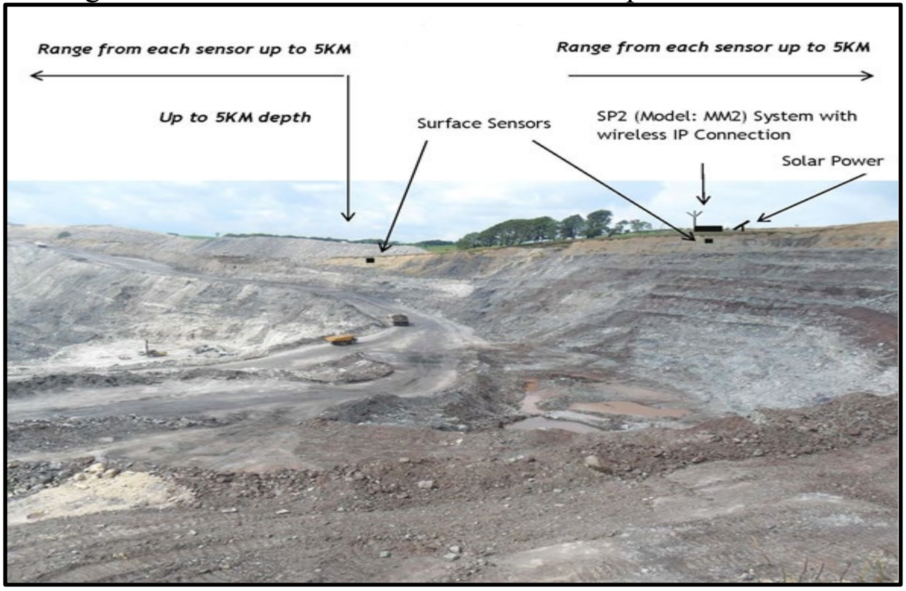
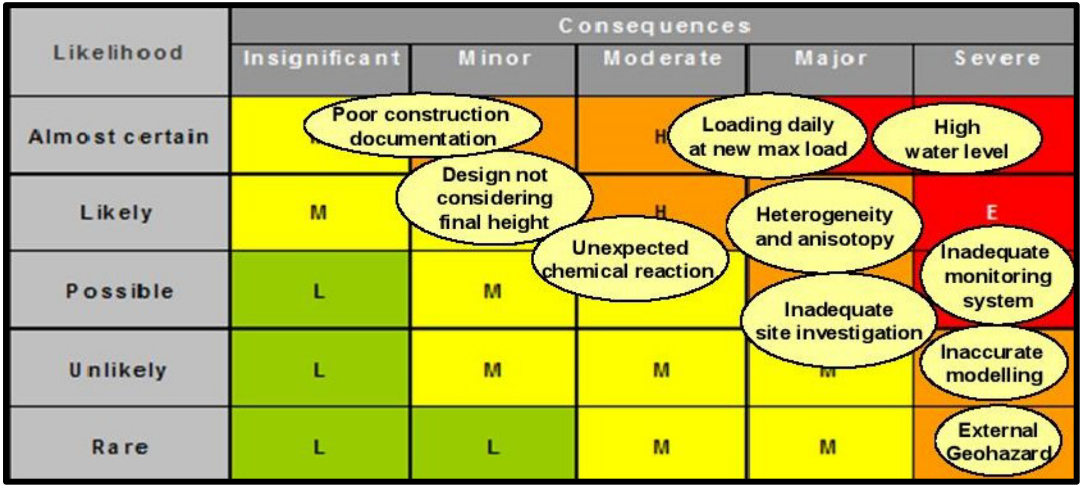

Range from each sensor up to 5KM
Range from each sensor up to 5KM
Up to 5KM depth
Surface Sensors
SP2 (Model: MM2) System with wireless IP Connection
Solar Power

Figure 13: An example of micro-seismic accelerographs
used for open pit and tailings dam monitoring (SureWave Technology Ltd.) [33]

7. Value VS. Cost of Tailings Dams Monitoring Systems

Tailings dams are more than 10 times more likely to fail than other conventional water retaining dams [34]. Operating a tailings dam involves risks that need to be identified, quantified and mitigated. These dams are subjected to many hazards that directly influence their stability, which need to be properly identified and assessed during both the dam construction stage and the following closure stage.

The main hazards potentially influencing the dam as to their likelihood and possible consequences are shown in a matrix form in Figure (14) [35].



Likelihood	Consequences				
	Insignificant	Minor	Moderate	Major	Severe
Almost certain			H		
Likely	M		H		E
Possible	L	M			
Unlikely	L	M	M	M	
Rare	L	L	M	M	

Poor construction documentation
Loading daily at new max load
High water level
Design not considering final height
Heterogeneity and anisotopy
Unexpected chemical reaction
Inadequate monitoring system
Inadequate site investigation
Inaccurate modelling
External Geohazard

Figure 14: Main risks affecting directly the dam stability [35]

Need for Monitoring and its extent comes naturally from the identified risks and need to mitigate safety conditions to prevent accidents and failures. So it follows that methods, types and extent of any safety monitoring program will be governed by the factors indicated in the aforementioned matrix.

Maintaining dam integrity requires ongoing focus on appropriate engineering design, quality construction and operating discipline. This is enabled through maintenance of the dam, good governance of its facilities; and may be followed through awareness of potential failure modes and continuous monitoring and surveillance followed by frequent safety reviews.

Need for a continuously running safety monitoring program is permanent throughout the whole life cycle of the tailings dam. The operational phase of any dam is dynamic and is likely to include expansion of the dam, raising of the dam height and/or addition of dams. The closure phase of a dam can often exceed in its duration the operational phase of the dam, yet it may need ongoing observation of seepage quantity and quality and their impact on surface and ground water, geochemical and physical changes that emanate from the inactive dam [36].

Failure of tailings storage facilities (TSF) can have disastrous consequences for nearby communities, the ecosystem, and for the mining companies, who may consequently face high financial and reputational costs.

The ecosystems impacts caused by any failure can last for many years depending on the nature of the tailings. As an example; In 2015, the breach of the Fundão Tailings Dam at Samarco mine in Minas Gerais resulted in 19 fatalities, and was declared the worst environmental disaster in Brazil’s history. In settlement of the claims resulting from this failure, the owners had to agree on a settlement plan costing $2.4 billion to remediate and compensate for the impacts over a 15 years. Moreover, twenty-one company executives were charged with qualified murder. In

The above mentioned example, the owning company is in the process now of restoring 5000 streams, 16,000 hectares of "Permanent Conservation Areas" along the Doce River basin, and1200 hectares in the riverbanks. It is estimated that the livelihoods of more than 1 million people were affected because of the failure [37].

The costs of installing, running and maintaining tailings dam safety monitoring program can be high, but not so high if losses of similar magnitudes as in the above case are to result.

The cost of any safety monitoring program may increase if modern real- time logging and transmitting are introduced. But the total cost, should be, in any case, be commensurate with the number of required measurements, types of instruments to used, and the method of data logging which are all functions of the dam risks and the consequences magnitude.

The running cost the system monitoring program, whether during the active phase of construction or through the inactive phase of closure, adds up to the initial cost, and the total budget of the program has to include provision for the safety review studies to analyst the collected data by highly qualified and highly paid consultant.

All these costs, however, can be justified in view of the large possible losses that the mining enterprise has to bear in case of failure. Any safety monitoring program can be, however, tailored according to the size of the tailing facility and is running operations.

In a great number of cases, the time value of money increases the pressure on private enterprise to maximize profits in the short-term.

Therefore, the required additional initial capital expenditure of installation and the ongoing cost of running an instrumented safety monitoring program may put these systems at disfavor, due to the continued managerial emphasis on deferring expenditure, promoting low-capital tailings disposal solutions with little emphasis on monitoring and rehabilitation costs.

8. Comments, Summary points and Conclusions

8.1 In the past 20 years' statistics show an increasing trend in tailings dam failures. This may have been a result of adverse weather conditions due to climate change and poor management of very large mining facilities. Moreover, rapid dyke construction following the need to satisfy the rising demand of growing industries over the mined metals has probably contributed in many ways to this end. In cases of absence of sound management procedures Increasing safety requirements of dams and more awareness of their increasing threats to the environment and societies have contributed to the development of more advanced monitoring means and methods. Although systems of monitoring tailings storage facilities are more advanced than conventional dams, even more accurate and automated, they are still not absolutely reliable, and many catastrophic failures were reported during the last decades.

The current engineering practice in tailings dam design is based on conventional dams' design principles, in which the selection of the appropriate magnitude of design loading is governed by the potential consequences, and the higher the consequence the higher the design loads used. The increasing magnitude of design loads also significantly increases the level of uncertainties in the stability analysis, material characteristics, its performance and the design load parameters themselves. The unique nature of tailings dams introduces additional uncertainties, which do not exist in water dams [38]. These facts plus the recent dangerous occurrences involving tailings dams support the need for establishment rigid and continuous dam safety monitoring program during both of its service and closure life.

8.2 Published statistics of tailings dams' failures and incidents are limited compared to those which belong to conventional dams and only major catastrophes have been reported. Most accidents pass without notice, either due to the remote locations of the mines, or because of the owners' desire not to publicize such events and keep a clean image in the eyes of the public, or even because their main concern is to keep production going without interruptions. The modes of failures of tailing dams in these cases show that many such accidents could have been avoided with good and responsible managements supported by rigid and continuous routines of safety surveillance and safety reviews. As in conventional dams this is achieved by visual observation and measurements, use of recording instruments, and analyzing all the accumulated data by competent engineers to come up with recommendation for changes and upgrading works. Monitoring program for tailings dams is often included in a comprehensive "Operations Manual". Operations manual is a document required by legislation in an increasing number of jurisdictions [39]. However, due to the lack of clear monitoring guidelines, some of them are a result of internal rules, local conditions and needs. Most of the mining companies have developed their own guideline for tailings dam safety

8.3 In spite of the different goals and methods of construction, tailings dams' surveillance requires all activities necessary for achieving safety and continuity of service similar to conventional dams, and should include all, and even more, of the ingredients as in conventional dams. Any useful safety monitoring program for tailings dams has to be a continuous chain of activities carried out in successive stages and extending over very long time which may last for decades. Dam safety programs for tailings dams differ from those applied in conventional dams in its duration. Such programs should cover the active life of dam construction, which goes on as long as the mine remains open for production, and its closure phase afterwards. Safety checks are normally comprised of routine visual inspection, use of recording instruments, performing various measurements, and so often use of automation to record and transmit the data to the control center for analysis and appraisal. Moreover, frequent safety reviews, carried out by professional engineers, have become mandatory requirement of Regulating Authorities in accordance with national legislations. Requirements of these legislation are normally commensurate with the risk category of the dam.

8.4 Recognition of the common failure modes of these dams show that they range between; rotational failures, foundation failures, overtopping, erosion of the slopes, piping and internal erosion, and liquefaction under seismic loading; which are also common in conventional earthfill dams. Parameters leading to these modes of failure include excessive rise of pond water level, high phreatic surface level and excess pore water pressure in the fill, development of seepages through fill and foundations, deformation of dam, settlement and differential settlement, total earth pressure and seismicity. Host of instruments, devices and methods for measuring and recording include, inflow discharge meters, piezometers, weirs, settlements plates, inclinometers, and geodetic surveys checking, while seismographs are important elements in any safety monitoring for dams especially in known aseismic active regions. Tailings dams offer very good opportunities for the use of modern real time measuring, transmitting and recording equipment in view of the large volume of such measurements and recordings required in very large facilities. In doing this, tailings enterprises, in spite of the high cost of such equipment, save a great deal in the cost of labor in a work which they consider not to fall in their mainstream activity. Moreover, such systems offer the opportunity of discovering an early threat before it is too late to interfere with repair or upgrading actions.

8.5 Major difference between conventional dams and tailings dams are the higher threats tailings dams present to the natural environment and the ecosystem. Most of the elements currently used in industrial applications, technology development, and energy generation are produced through the mining operations and disposal of the waste in tailings dams and ponds. Adverse changes in the landscape can result from the high demand of water and energy and the large quantities of mobile effluents associated with tailing production and disposal. Historically, this activity can be considered as one of the main anthropogenic activities that contribute to major and trace elements pollution of river basins worldwide, which may lead to serious human health implications and also long-term impairment to waterways and biodiversity. There are two serious problems usually associated with mining activities, the acid-mine drainage and the mine tailings that contribute deliberately and/or accidentally to the burden of anthropogenically derived metals to river basins.

What differs tailing dam accidents from other anthropogenic activities is the amount and velocity which slurry may travels, covering soils, river sediments, floodplains, riverbanks, and damaging water quality and adversely impacting plant and animal life. Taking all this into consideration warrants looking into the question of establishing a good and robust safety monitoring program seriously. Any cost- benefits analysis of such program should be based on the assessment of the risks involved in tailing spills due to accidents or complete failures. Investments in such program have to be matched against the high costs of compensations, rehabilitation and restoration of the damaged environmental which many take generations after the closure phase. In summary establishing and running such a program is highly recommendable, and it may be considered as part of the insurance cost of the safe running and management of this industry.

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Geophysical Methods and Their Applications in Dam Safety Monitoring

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Abstract

The use of geophysical methods in dam sites investigations and safety monitory has proved their good value and versatility in many earthfill dam sites as early as the 1920s. In the following years great development has occurred in the methods, application procedures and tools used. They may be considered today as good ways for carrying out observation tasks on existing dams in non- intrusive and much faster and cheaper ways than the traditional geotechnical methods. It is possible using them to discover anomalies in the dam body or its foundation at an early stage and allowing quick intervention repair works. These methods seek to register and present variations in the basic geotechnical material properties in dams such as; bulk density, moisture content, elasticity, mechanical properties of rocks, electrical resistivity and mineralogy and magnetic properties and so forth. Such variations can indicate increasing seepage flow, progression in cracks' sizes, formation of voids, caverns and other instability manifestations. Depending on how any investigation is carried out and the targeted anomaly, there is now selection of these methods such as; Electromagnetic Profiling (EM), Electrical Resistivity Tomography (ERT), Self-Potential (SP), Ground Penetration Radar (GPR), variety of Seismic Methods (SM) which can be applied using such equipment as in Seismic refraction, Seismic Reflection, Multi Analysis of Rayleigh surface waves (MASW) instruments, or using Refraction Micrometer (ReMi), macro-gravity method, and Cross- Hole Seismic Tomography. In addition to Temperature Measurements, and other less used methods for Microgravity measurement, Magnetic Profiling and Radio Magnetotelluric methods. An attempt is made here to cover the details of these methods, their advantages and limitation and to prove their usefulness in many dam sites all over the world. One observed issue is their adaptability to embankment

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dams more than to concrete dams and their popularity for checking seepage related problems and material changes within dam bodies and their foundations such as formation of voids and sinkholes.

Keywords: Geophysical Methods, Geotechnique, Electromagnetic Profiling, Electrical Resistivity Tomography, Self- Potential, Ground Penetration Radar, Seismic refraction, Seismic Reflection, Rayleigh Multi analysis of surface waves, Refraction Micrometer, Cross- Hole Seismic Tomography; Refraction Micrometer.

1. Introduction

Geophysical methods have the potential to offer a comprehensive and cost-effective way of assessing the performance of dams and dikes non-intrusively that is complementary to other surveying techniques and can be applied across large areas of sub-surface. There is a clear need for detailed assessment of embankment dams and dikes in terms of their internal structuring and integrity. While the vast majority of dam inspections, whether visual or instrumental, consists of checking for external indicators that an issue exists within the dam or dike such as cracking, movements or increased seepage flow, these geotechnical forms of assessments may not identify an issue that is driven by internal processes until they are in progression. More detailed studies are sometimes carried out in the form of geotechnical investigation, but geotechnical methods are intrusive and costly when used over an entire site.

It is interesting to see nowadays that geophysical methods play an increasing and important role in many dams and dikes' projects in the world in the same ways as other geotechnical investigation methods. In both geotechnical and geophysical methods, the objectives under consideration are the same, as both aim towards characterizing the soil, rock and groundwater at dam potential sites and examination of anomalies in existing dams. The targeted mechanical properties of soil and rock in both types of investigation are; shear modulus, bulk density, porosity, as well as the lithology and patterns of fracturing and weathering, in addition to fracture orientation, depth to bedrock, and fault location. Some geophysical methods, however, are superior to other geotechnical methods in that they can still be used to observe water seepage flow and formation of voids or sinkholes and can offer this at a much cheaper cost. These methods range between techniques such as Electromagnetic Profiling, Electrical Resistivity Tomography, Ground Penetrating Radar, Seismic Methods Profiling, Seismic Methods, Temperature Measurements and others. The selection of any one or combination of number of them to be used in one site depends on many factors which will be discussed later, but it may be said that all these methods are well established now for these applications and there are many other techniques gaining in popularity.

Dams are man-made structures constructed as barriers to hold water for various uses, including water supply, irrigation and flood control. In embankment dams, the embankment is usually constructed with different materials including earth materials (soils), rock boulders and concrete. The embankment resists the hydrostatic forces of water in the reservoir and in the majority of cases must contain a section or zone impervious enough to prevent excessive seepage. Thousands of such dams have been built all over the world with various shapes and materials configurations and on different foundations. Geophysical methods have found their applications in different situations in these dams and they have been used to evaluate post-construction changes within and beneath dam embankments.

2. Historical Development of Geophysical Methods and their Application to Dams

Electrical geophysical methods were the first of such methods in the field of dams. Electrical prospecting has grown from Fox's investigations of natural earth currents in 1830, through Schlumberger's successful use of applied direct currents, to a diversified art employing both alternating currents and electromagnetic fields, as well as direct currents. Early resistivity data were interpreted by empirical methods, which are still used widely. Other interpretative methods include direct interpretation by transforming the resistivity data into geologic information using mathematical transformation formulas [1].

Logical and mathematical interpretation of resistivity data application of engineering geophysics for investigations was recognized as early as 1928, when Crosby and Leonardon (1928) applied electrical methods successfully to map bedrock topography at a proposed dam site on the upper Connecticut River in what was apparently the first engineering application of these methods in the United States. The bedrock consisted of a high-resistivity Precambrian schist overlain by about 150 feet of glacial drift; the resistivity contrast was about 10:1. The Schlumberger method of taking a vertical profile about a single electrode was used. Of the results of the measurements at eight holes where tests were made prior to the full-scale survey, five gave accurate depths within 5 percent and the others were within 20 percent [2].

Electrical resistivity profiling method the distance between the voltages probes is (a) and the distances from voltages probes and currents probes are (c) while the depth of penetration of the probe is (b); as shown in Figure (1), then using the Schlumberger method, if (b) is small compared to a and c, and c>2a, then the apparent soil resistivity value can be calculated. Equation and method of calculation are given in reference [3], and this is further discussed in the following paragraph (4).

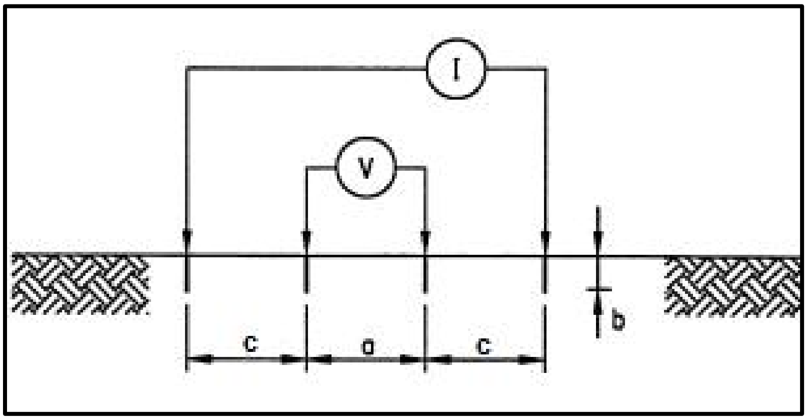


Figure 1: Diagram showing Schlumberger set up for electric resistivity measurement [3].

Following 1928, geophysical methods became part of the investigation programs for potential dam sites, but further growing of geophysical applications on existing dams mainly focuses on the operation phase after the dam is complete.

Dams are massive structures and their internal hydraulic conditions may require attention before problems are detected by simple reconnaissance methods, visual inspections and instrumental monitoring. These methods do not provide information on the dynamics of what is happening inside the dam, and the discrete monitoring instruments provide engineering parameters with limited spatial coverage of the dam. There is now, therefore, a growing demand for the non-destructive geophysical techniques to internally image the dam for early detection of anomalous phenomena and facilitating early remedial actions.

Geophysical techniques, such as time domain reflectometry, travel time velocity tomography, electrical resistivity tomography, and multi-station analysis of surface wave, have advanced significantly in the last few decades and application of these methods on dams have become of great interests to engineers. A better understanding of the connections between geophysical results and engineering significance related to dam safety and sustainability can help engineers gain more useful information when employing these technologies.

In a recent article (2018) limited number of cases from Asia, America, and Europe were gathered and listed in Tables (1,2,3) respectively [2]. The details of each of each case may be obtained from the reference given in the last column of the tables. These tables are not comprehensive as many other case studies do exist and more of them can be found all over the world which are either not discussed in the relevant conferences and workshops or their reports are not published. These tables, however, are intended to show the application trend nowadays, and major problems to which geophysical methods can be applied. Among these cases, it can be seen that abnormal seepages in earth dams draw most of the attention of geophysical groups. Internal erosion is the top safety concern in earth dams, and abnormal seepage is the observable symptom of it. However, depending on the source and seepage path, not all the abnormal seepages result in internal erosion.

Electrical methods such as electrical profiling, electrical resistivity tomography (ERT), and self- potential (SP) method have been recognized as water-sensitive technologies and used to investigate the spatial distribution of wetted area and possible flowing paths.

Self -potential or spontaneous potential (SP) is a naturally occurring electric potential difference in the Earth which can be one of different electrical potentials; i) electro- kinetic, or streaming, potential, due to the flow of a fluid with certain electrical properties passing through a pipe or porous medium with different electrical properties, ii) liquid-junction, or diffusion, potential caused by the displacement of ionic solutions of dissimilar concentrations, iii) mineralization, or electrolytic contact potential produced at the surface of a conductor in contact with another medium, and, iv) Nernst potential which occurs when similar conductors have a solution of differing concentrations about them.

The other major application of the geophysical methods other than seepage is the investigation of cracks or voids in dams’ foundations or bodies. Cracks and/ or voids in dams create preferential flowing paths susceptible to further erosion evolution. Other developed methods such as Ground penetrating radar (GPR) and seismic tomography (ST) have gradually become are popular technologies for such purpose. If the voids or cracks are close to the surface, (GPR) may be an effective tool to quickly map their locations and depths. On the other hand, if the voids or cracks are too deep then Self- Potential (SP) is a good alternative since it is difficult to detect small voids or cracks by seismic methods. The concept of applying (ST) here is not to directly locate them, but to search for low velocity anomalies caused by the diffraction around the voids or cracks. The diffraction would increase the ray path and hence reduce the estimated velocity.

It may be said that current geophysical techniques used in embankment dams and levees are generally sensitive to the distribution of the bulk “geophysical” properties (elasticity, electrical resistivity, dielectric constant, density, etc.) in the subsurface that are in turn related to more “basic” properties (bulk density, water content, porosity, mineralogy, etc.). All these methods are used by engineers, including the USACE and other dams’ owners for various geophysical assessment of dams and levees [19].

In summary, it may be said that available literature on the geophysical applications indicates an increasing trend during the past few decades in using geophysical techniques and especially for dams’ safety investigations.

Table 1. Case studies of applying geophysical methods for safety assessments of dams in Asia.

Dam	**Aim of investigation**	**Applied geophysical method**	**Country**
Shuishe Dam	Abnormal seepage in downstream shell	2D & 3D ERT	Taiwan Taiwan Power Co. Files
Hsinshan Dam	Abnormal seepage in downstream shell	Time-lapse 2D ERT Lin	Taiwan
Wushantou Dam	Slips of slope in downstream shell	MASW	Taiwan
Shigang Dam	Strength of concretes after chichi earthquake	Seismic tomography	Taiwan
Xishi Dam	Aging concrete	Seismic tomography	Taiwan
Sandong Dam	Abnormal seepage in dam abutment	2D ERT; SP	China
Unknown Dam in Korea	Abnormal seepage in downstream shell	2D ERT Cho and Yeom (2007)	Ref. [4]
Afamia B Dam	Abnormal seepage in dam foundation	EM; Electrical profiling; 2D ERT	Syria [5]
Som-Kamla-Amba Dam	Abnormal seepage in downstream shell	SP; Electrical profiling	India
Nanshui Dam	Voids inside dam body	GPR	China
Sanqingting Dam	Cracks in dam body	GPR	China
Ribeiradio dam,	Characterization and treatment evaluation of dam rock mass foundation	Time-lapse crosshole seismic tomography	Portugal Ref.[6]
Akdeğirmen Dam	Abnormal seepage in downstream shell	2D ERT	Turkey Ref. [7]

Table 2. Case studies of applying geophysical methods for safety assessments in American dams.

Dam	Aim of investigation	Applied geophysical method	Ref.
Dana Lake Dam	Abnormal seepage in dam body	SP; electrical profiling	[8]
Amistad Dam	Abnormal seepage in dam foundation	2D ERT	[9]
Barker Dam	Voids in dam body and seepage	SP GP	[10]
Avon Dam	Abnormal seepage in downstream toe	SP; 2D ERT GP	[11]
Cordeirópolis Dam	Abnormal seepage in dam body	2D ERT	[12]

Table 3. Case studies of applying geophysical methods for safety assessments in European dams.

Dam	Aim of investigation	Applied geophysical method	Ref.
Mornos Dam	Abnormal seepage in dam body	2D ERT, Seismic Tomography	[13]
CHB Dam	Abnormal seepage in dam body	2D ERT, 2D ERT	[14]
EI Tejo Dam	Abnormal seepage in dam body	SP	[14]
Hällby Dam	Abnormal seepage in dam body	2D ERT monitoring	[15]
IJKdijk test Dam	Internal erosion in dam body	SP; Acoustic emission	[16]
Monte Cotugno Dam	Abnormal seepage in dam body	2D ERT monitoring	[17]
Vitineves Dam	Abnormal seepage in dam body	EM, 2D ERT; SP	[18]

3. Monitoring Seepage in Earthfill Dams

Dam failures world record shows that many major earth dam failures are due to seepages. Seepage quantity if within the design limits, and if its quality does not indicate internal erosion progression, then the stability of an embankment dam is not threatened, but continuous excessive seepage of turbid water will eventually lead to dam failure. The combined Electrical Resistivity Tomography (ERT) results and geotechnical monitoring data can give good indicators of likely mechanism for abnormal seepages. Most of the available seepage geotechnical monitoring instrument systems are insensitive to small changes in the seepage flow, however, the non-intrusive geophysical techniques have been on high demand to facilitate early detection or diagnosis of anomalous seepage occurrences. The ERT results may be combined with geotechnical data to clearly indicate the seemly mechanism of abnormal seepage.

Seepage anomalies observation can be enhanced in its progression by time-lapse measurement. In many cases the (2D) electrical resistivity tomography was used to image suspected seepage channels in an earth dam. This method was used to diagnose variations in the water content amount in dams, whether it was result of an anomalous seepage at the subsurface or, mere seasonal effect. Various zones of relatively uniform resistivity values were mapped and identified from the interpreted acquired data. The results revealed that the rather low resistivity zones within the bedrock represent probable pathways for seepages. Hence, geophysical methods have been successfully used to detect and map the seepage pathways within the bedrock of earth dam foundations.

Al-Fares (2011) carried out surveys involving various geophysical methods such as the superficial electromagnetic (EM) method, electrical sounding and (ERT) to identify and discern the cause of the water losses problem in the Afamia B dam located at Al- Ghab basin in Syria. He noted that the (ERT) technique was more effective and convenient to detect water infiltration problems and leakages through the bedrocks. The integrated results of the methods revealed some geological structures that have adverse effect which may eventually lead to leakages in the dam foundation. The vertical infiltration in the dam was later detected as the principal cause for the seepage problem. His final conclusions from the revealed results were; i) the presence of some geological structures which have negative role and led to essential leakage in the dam, ii) the ERT was an efficient technique in outlining detailed subsurface geological features such as fractures or faults; this technique has succeeded in detecting some underlying anomalies which has been confirmed by excavating a trench and that, and iii) the vertical infiltration constitutes the principle cause for the seepage problem in the dam itself [20].

Monitoring of seepages around embankment dams is very important in civil engineering hydraulic structures. Generally, the effects of seepage on dams are functions of soil water retention capability which is a vital factor in engineering geology. Geological mapping and field investigations for dam projects should delineate soil properties and rock formations that could cause seepage failures, and an in-depth study of the geological and geotechnical properties of these materials should be carried out. A combination of (EM), (ERT) and other tools such as (SP), have proven to give quantitative and qualitative interpretation of seepage flow. Performing resistivity and self-potential profiles along several lines across the dam permits obtaining a quasi-three dimensional view of the seepage pattern [21].

It has been concluded from many study cases that different geophysical methods can supplement each other in characterizing and identifying subsurface seepage areas as they relate to the bedrock fracturing, culturally emplaced drainages, and variations in soil type.

Geophysical methods have been proven to detect, map, and monitor seepage areas within earthen embankments. Ideally, investigations should include more than one method and then the results should be tied into existing borehole data to provide the best seepage characterization results. Geological data from the boreholes give a context to data interpretation by defining what geologic features are possible or likely [22].

Lastly, it may be said that geophysicists and civil engineers should work hand in hand in order to build and maintain dam structures to avoid the damage and waste of resources that may eventually result in the project if done otherwise.

4. Principles of Electric Resistivity Method

Electrical resistivity imaging is a widely used tool in near surface geophysical surveys for investigation of various geological, environmental and engineering problems including landslide and earthfill dams and levees.

The relevance of electrical surveys is to identify the subsurface resistivity distribution by making measurements on the ground surface. The true resistivity of the subsurface can be quantified from these measurements. The electrical resistivity is related to various geological parameters such as the mineral and fluid content, porosity and degree of water saturation in the rock or within a fill dam. Electrical resistivity surveys have been used for many decades in hydrogeological, mining and geotechnical investigations of engineering structures. More recently, it has been used for environmental surveys and for dam safety monitoring.

As already explained in the previous Paragraph (2), resistivity measurements are normally made by injecting an electric current into the ground through two electrodes (C1 and C2), and measuring the resulting voltage difference at two potential electrodes (P1 and P2). From the current (I) and voltage (V) values, the resistivity (pa) value is calculated as given below,

$pa = k V / I$,

where (k) is the geometric factor which depends on the arrangement of the four electrodes.

Resistivity meters normally give a resistance value $R = V/I$, so in practice the apparent resistivity value is calculated by

$pa = k R$

The calculated resistivity value is not the true resistivity of the subsurface, but an “apparent” value which is the resistivity of a homogeneous ground which will give the same resistance value for the same electrode arrangement. The relationship between the “apparent” resistivity and the “true” resistivity is a complex relationship and inversion of the measured apparent resistivity values can be made by mathematical equations which has been facilitated lately by using computer programs.

The resistivity method has its origin in the 1920’s due to the work of the Schlumberger brothers and Wenner, and for approximately the next 60 years it was used for quantitative interpretation, conventional sounding surveys.

In this method, the center point of the electrode array remains fixed, but the spacing between the electrodes is increased to obtain more information about the deeper sections of the subsurface. The resistivity method measures the apparent resistivity of the subsurface, including effects of any or all of the following: soil type, bedrock fractures, contaminants, and ground water. A change in electrical resistivity may suggest changes in composition, layer thickness, presence of voids, presence of seepage or pollution levels, and such measurements are also useful for detecting lateral and vertical changes in subsurface electrical properties at the same time.

Electrical resistivity imaging is therefore, a geophysical prospecting technique designed for the investigation of areas of complex composition or geology; this involves measuring a series of resistivity profiles by changing the spacing’s between the electrodes. Since increasing separation between electrodes provides information from increasingly greater depths, the measured apparent resistivity can be processed to provide an image of true resistivity against depth.

The principal applications of this technique are; definition of aquifer boundaries, mapping bedrock configurations, identifying faults and fractures, locating karstic voids in the ground, and in embankment dams, the identification of seepage paths and other material change anomalies.

Majority of rock and soil minerals are not good electrical conductors although some native metals and graphite conduct electricity well. The measured resistivities in earth materials are mainly controlled by the movement of charged ions in pore fluids. Even though water itself is not a good conductor of electricity, ground or seepage water in most cases contains dissolved compounds that greatly improve its ability to conduct electricity. Hence, porosity and fluid saturation tend to dominate electrical resistivity measurements. Cracking and voids within crystalline rock or earthfill dam body can lead to low resistivities if they are filled with fluids. Resistivity values of the various earth materials are shown in Table (4) [23]. A more representative form of these values is given in the diagram in Figure (2) available from references [24] [25]. In this diagram the resistivity of soils and rock are shown to vary over a very wide range of differences, where these differences form the basis of resistivity surveying. Moreover, it is essential to be aware of the large overlaps in the resistivity values of different types of earth materials. As a result, measured resistivity should never be interpreted directly as a certain material category without additional knowledge of the specific situation.

Table 4: Electrical Resistivity values for different types of soils [23]

Type of Soil	**Mean value of Resistivity in Ωm**
Swampy soil, bogs	1- 10
Silt alluvium	20 100
Humus. Leaf mould	10- 50
Peat, turf	5- 100
Soft clay	50
Marl and compacted clay	100- 200
Jurassic marl	30- 40
Clayey sand	50- 500
Siliceous sand	200- 300
Stoney ground	1,500- 3,000
Grass- covered – stony -sub-soil	300- 500
Chalky soil	100- 300
Limestone	1000- 5000
Fissured limestone	500- 1000
Schist, shale	50- 300
Mica, schist	800
Granite and sandstone	1,500- 10,000
Modified granite and sandstones	100- 600
Fertile soil, compacted damp fill	50
Arid soil. Gravel, uncompacted non- uniform fill	500
Stoney soil. Bare, dry sand , fissured rocks	3,000

The conduction process of electrical current in geological materials is mainly electrolytic, and most of the common soil and rock forming minerals are insulators in the dry state. It is therefore logical to say that the amount, the distribution and properties of water present largely determine the resistivity of any soil or rock mass. For rock mass this means that fractures, faults and shear zones constitute the dominating pathways for electrical current, whereas the solid rock normally is considered as an electric insulator. An exception is rock with metallic cement that may allow significant conduction through crystalline structure.

Soils on the other hand, are porous media consisting of solid skeleton of particles, or grains, with pores in between. The grains are considered electrical insulators and the conduction is concentrated in the pore spaces that are typically filled or partly filled with water. Therefore, the resistivity of soils is strongly influenced by the amount of the present water, which in its turn is determined by the porosity and the degree of saturation; a property utilized in visualizing seepage flow in earthen dams and their foundations, in addition to recognizing signs of internal erosion.

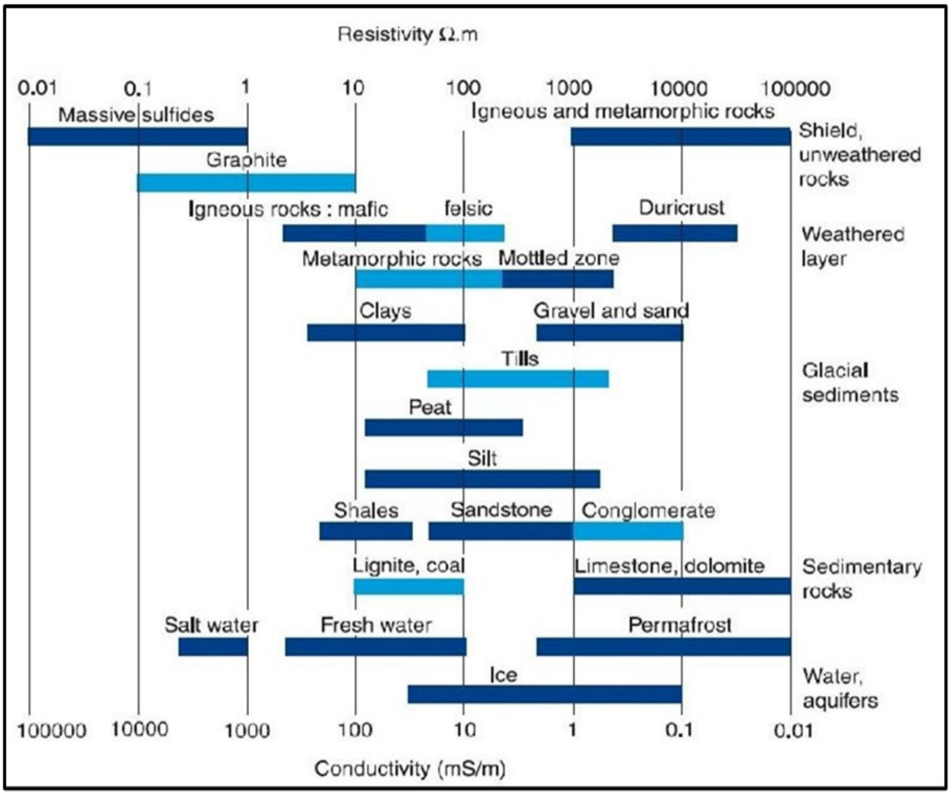


Figure 2: Resistivity (and its inversion, conductivity) of the main earth materials (Palacky 1991) [25].

In embankment dams and dikes the main aim of resistivity measurement is to verify their safety and integrity. This is done by detecting potential weaknesses, such as defective zones, anomalous seepages or internal erosion process. In such investigations, resistivity profiling and/ or resistivity soundings have been commonly performed but in recent years 2D and 3D approaches have been applied.

In most dam investigations the resistivity method has been used together with various other geophysical methods, which emphasize the importance of integrated multiple- method programs and also suggest the possible use of a monitoring strategy, where geophysical anomalies detected as a function of time can be correlated with reservoir water level.

Repeated and regular measurements on embankments over time have been performed in some cases as a way of continuous safety surveillance. One example for such case can be cited from the Swedish research program that was carried out for resistivity monitoring of Hälsby embankment dam by daily measurements in which daily measurements commenced in 1996 and five years later, in 2011, measurements were started on a second dam; the Sädva Dam [26].

Dam geometry considerations and reservoir water level variations can bring certain complications in dams' resistivity values interpretation when such measurements are made for checking the integrity of such dams, or for detecting anomalous seepage in the dam body or its foundations. Considerable research over the use of (2D) or (3D) resistivity surveying and sources of errors in these procedures has been conducted to solve such complications.

5. Suitability and Limitations of Geophysical Techniques

Commonly used geophysical methods in dam safety surveillance studies can be one or combination of many of the techniques already mentioned in the previous paragraph, i.e. Electromagnetic Profiling, Electrical Resistivity Tomography, Ground Penetrating Radar Profiling, Seismic Methods, Self-Potential Methods, Temperature Measurements and others. Selection of any one of these techniques depends on;

i. the orientation and objectives of the required study and the anomaly under consideration, whether seepage flow intensity, seepage path location, occurrence of internal erosion, formation of cavities and sinkholes, variation in material strength and so on.
ii. the environment surrounding study site and possibility of interference or attenuation by various external sources. Rural and coastal areas typically have much fewer sources of noise than urban environments. The majority of reservoir impounding dams are found in rural areas, and coastal areas are mainly characterized by the presence of salt water, so the types of structures found there are sea walls, piers and the other sub-types defined as beach structures and aids to navigation. Urban environments could also refer to urban/coastal environments as densely populated areas located by the sea. Table (5) delineates each environment type by the likelihood of the presence of factors that would impede certain geophysical techniques [27].

Table 5: Environmental Sources of Interference .

	Environment		
Limiting Factor	Rural	Coastal	Urban
Metallic disturbances	Possible	Possible	Likely
Potential Electromagnetic source	Possible	Possible	Likely
Vibrations from traffic etc	Unlikely	Unlikely	Likely
Water Type	Fresh	Salt	Fresh/Salt

iii. Available time and budget allocations as these two factors can heavily influence decision making in selecting the most suitable technique. Each geophysical technique, therefore, must be analyzed in terms of its time efficiency and cost effectiveness.

Various aspects related to geophysical methods applications are summarized in the following tables; table (6) shows the applicability of the various geophysical methods to earthen structures and their limitations with possible sources of errors; table (7) illustrate suitability of these methods to various detection requirement, while table (8) indicates the adaptation of these methods to type of detection required, ease of application, required depth of investigation and resolution and details, ease of processing and limitations with respect to available time and budget constraints.

Each of the geophysical methods has some aspects which make them more suitable to follow a particular issue or presents some limitations as can be seen from these tables. It is well understood that geophysical methods generally are useful as non- destructive remote sensing tools that can provide information over large volumes as compared to point measurements. However, the anomalies that are associated with internal erosion in embankment dams often are very small, and the effectiveness of geophysical techniques to detect changes in seepage conditions is improved through repeating surveys or adopting a long-term monitoring approach. In addition, application of more than one geophysical technique will provide added confidence in the interpretation and detection of anomalous features.

Complex inter-relationships exist between various parameters, such as water content, porosity, total dissolved solids, mineralogy, temperature, electrical resistivity, coupling coefficient, and self- potential (SP). Not recognizing some of the fundamental relationships and carrying out a one-time survey without supporting information could lead to misleading and often disappointing results.

Table 6: Geotechnical methods: Types; their suitability and their limitations [27]

	Structure		Limitations due to Location
Geophysical Method	Dikes, Levees (Transient Seepage)	Impounding Dams(Steady State Seepage)	Rural, Costal, Urban
Electromagnetic Profiling	EM- 31, handled by 1 Person, Very fast data collection	EM- 34, 2 persons operated, compact fast data collection	Suitable for rural and coastal sites. Not suitable in urban sites where too many metallic disturbances.
Electrical Resistivity Tomography	Not time effective to be done across km	Suitable	Suitable for rural and coastal sites. Great care must be taken to account for metallic objects in urban sites.
Ground Penetrating Radar Profiling	Soil dependent, Ground must be reasonably free of vegetation etc.	Depth of Investigation too low for monitoring down to foundation soil. Soil dependent	Suitable for rural and urban sites. In costal sites it may be suitable but method is soil dependent and salt water.
Seismic Methods	Not time effective to be done across km	Suitable	Suitable for rural and coastal areas. In urban areas sources of traffic and other vibration must be removed
Self- Potential Methods	Only during flood conditions	Suitable	Suitable for rural sites. Soil water can disrupt data in coastal sites. In urban sites device is very sensitive to electrical perturbations
Temperature Measurements	Only during flood conditions	Suitable	Suitable for all sites

Table 7: Geophysical Methods and their suitability for targeted objective [27]

	Detection Requirement objectives				
Geophysical Method	Internal Composition/ zoning	Identifying seepage paths	Seepage flow intensity	Large voids due to animal borrowing, erosion cavities	Damage from environmental desiccation, root growth etc
Electromagnetic Profiling	Suitable only as 1 D mapping	Suitable	Not suitable	Suitable	Could be detected but not identified solely from EM
Electrical Resistivity Tomography	Suitable	Suitable	Not suitable	Suitable	Suitable
Ground Penetrating Radar Profiling	Suitable	Not suitable	Not suitable	May not resolve smaller animal borrowing	Suitable
Seismic Methods	Suitable	Not Suitable	Not Suitable	Suitable	Not suitable
Self- Potential Methods	Not suitable	Suitable	Suitable	Not suitable	Not suitable
Temperature Measurements	Not suitable	Suitable	Only suitable with repeated measurement	Not suitable	Not suitable

Table 8: Summary of analysis of geophysical techniques [27].

Geophysical Method	Type of Detection	Field Implementation	Depth of investigation (DOI)	Data processing resolution	Limiting Factors	Acquisition speed	Cost
Electromagnetic Profiling	Overall dam assessment, soil properties through soil electrical conductivity.	Easiest data gathering method.	Variable, smaller equipment around 3m, larger down to 40- 50m	Low resolution but straightforward to process.	Not detailed enough for use as the sole monitoring technique.	Fast	Low
Electrical Resistivity Tomography	Detailed assessment, soil properties and geological structure, seepage and void detection.	Can be problematic. Applicable in the largest variety of circumstances.	Site and configuration dependent but quite large.	High resolution, needs corroboration for other data	Must account for metallic objects and other artifacts in data.	Slow	High
Self- Potential Methods	Detects soil conditions and seepage paths as well as seepage intensity.	Land or Waterborne. Simple implementation.	Waterborne. DOI large enough for most reservoirs	Medium resolution. Interpretation must account for any other sources	Very susceptible to interference. Not applicable for salt water/ “dry” sites.	Slow	Medium
Ground Penetrating Radar Profiling	Fairly detailed soil properties, material changes, internal structures and voids.	Straightforward can be towed manual or vehicle.	Small, generally no more than few meters.	Good resolution but very complex interpretation	Not applicable for low resistivity materials- clayey for example.	Fast	Low
Seismic Methods	Detailed. Mechanical properties of soil/material, structures/voids.	Reasonably easy to implement.	Large enough to reach foundations.	Requires high level of expertise in processing/ analysis. Low-Medium resolution	Time consuming. Soil characteristics may inhibit DOI.	Slow	Very high
Temperature Measurements	Anomalies in the seasonal temprature gradient can indicate seepage.	Straightforward installation but can be expensive.	Intrusive- any depth to which sensors can be positioned.	Interpolation of discrete values. Resolution dependent on prob.	Sounding method is intrusive. Needs distinct temp-gradient contrast between water and medium.	Medium	High

Geophysical techniques applied to the detection of seepage and internal erosion in embankment dams are at various stages of development even now. Of such late developments are the Thermal or Temperature Geophysical Measurements Method.

Temperature measurements appears to be now as one of the most-developed and best-understood techniques. With the recent advances in improved accuracy and resolution in measuring temperatures along fiber optic cables, there are exciting future possibilities.

For dam safety applications, the self- potential and resistivity methods generally appear to hold more promise than seismic methods as non-intrusive techniques applied at the surface of a dam. However, in specific settings, cross-hole seismic techniques could prove indispensable.

It is imperative that dam owner and practicing engineer should recognize the limitations and care required in planning, executing, and interpreting the results of all geophysical surveys. Geophysical data interpretation is non-unique and should be constrained by incorporating all available site information and integrating the interpretation of complementary data sets. Thus, strong cooperation between the geophysicist and engineer is essential to improve the interpretation and usefulness of the results [28].

6. Technical details of the geophysical methods

From the wide range of geophysical methods discussed already, it is clear that these methods are non- intrusive and fairly cheaper than the other conventional geotechnical methods involving visual observations and instrumental monitoring.

These techniques, however, need in the majority of cases supporting data from the other conventional methods and from other geotechnical investigation results for the proper interpretation of the obtained data to remove doubts or confusions.

Moreover, geophysical techniques can be used for ascertaining the safety of embankment dams and dikes when additional quick data are needed.

In most cases, experience up to now shows that these methods have been used to varying degrees of success to check the integrity of embankment dams that might have been affected by development of new seepage paths in the dam itself and/ or its foundation and development of excessive seepage, or to observe changes in the material properties and development of dangerous cavities or sinkholes. Repeated application of such methods on the same structure at uniform time intervals can give a true picture of the progress of such anomalies with time and allows decision taking for necessary repair works when needed. More technical details on the application and methodologies are given hereafter for better explanation of these techniques.

6.1 Electromagnetic (EM) Profiling

Electromagnetic profiling (EM) method is used to measure conductivity differences of geologic material. It provides data on soil properties through soil electrical conductivity. In the case of seepage studies, possible seepage paths can be located through the identification of high or low conductivity anomalies, where water-filled or clay-filled features can produce high-conductivity anomalies and air-filled features can produce low-conductivity anomalies. Subsurface soil types can also be inferred from electromagnetic measurements in addition to changes in structural damage, and the presence of buried metallic objects. However, other factors such as porosity, degree of saturation, and temperature can also affect conductivity measurements.

In electromagnetic (EM) surveying, the electrical conductivity of the ground is measured as a function of depth and/or horizontal distance. Different rocks, buried structures and objects exhibit different values of electrical conductivity. Mapping variations in electrical conductivity can identify anomalous areas worthy of further geophysical or intrusive investigation.

For shallower depths, data is gathered using a handheld boom-shaped device holding an Electromagnetic field transmitter on one end and a receiver on the other.

This can be operated by one person traversing along the surface of the embankment as shown in Figures (3, 4). For greater depths, two person systems exist which are still very straightforward to implement.

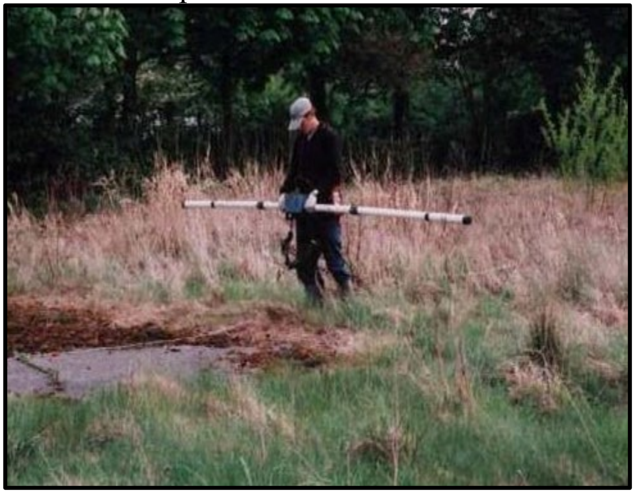


Figure 3: Acquisition of EM.31 data instrument [29].

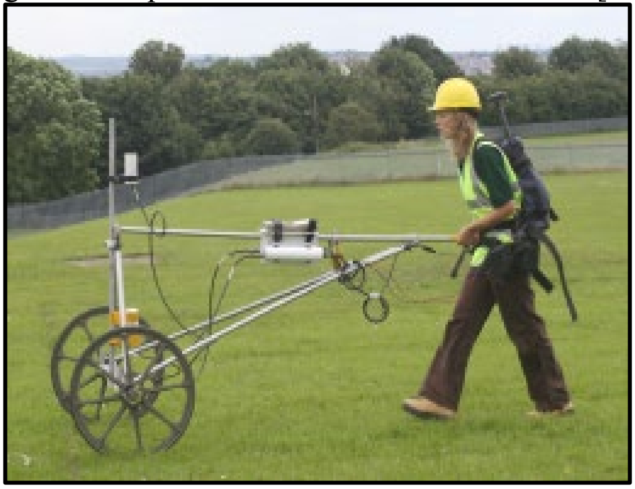


Figure 4: Acquisition of EM data instrument [30]

The EM instrument system consists of an electromagnetic loop transmitter and a loop receiver where the transmitter generates a primary electromagnetic field that propagates above and below ground. When the primary (EM) field encounters a conductive material within the subsurface, alternating currents occur which, in turn, generate their own secondary (EM) field. The receiver detects the secondary EM field along with the primary field that travels through the air as shown in Figure (5). The ratio between the secondary and primary (EM) fields provides a comparative reading of the apparent ground conductivity.

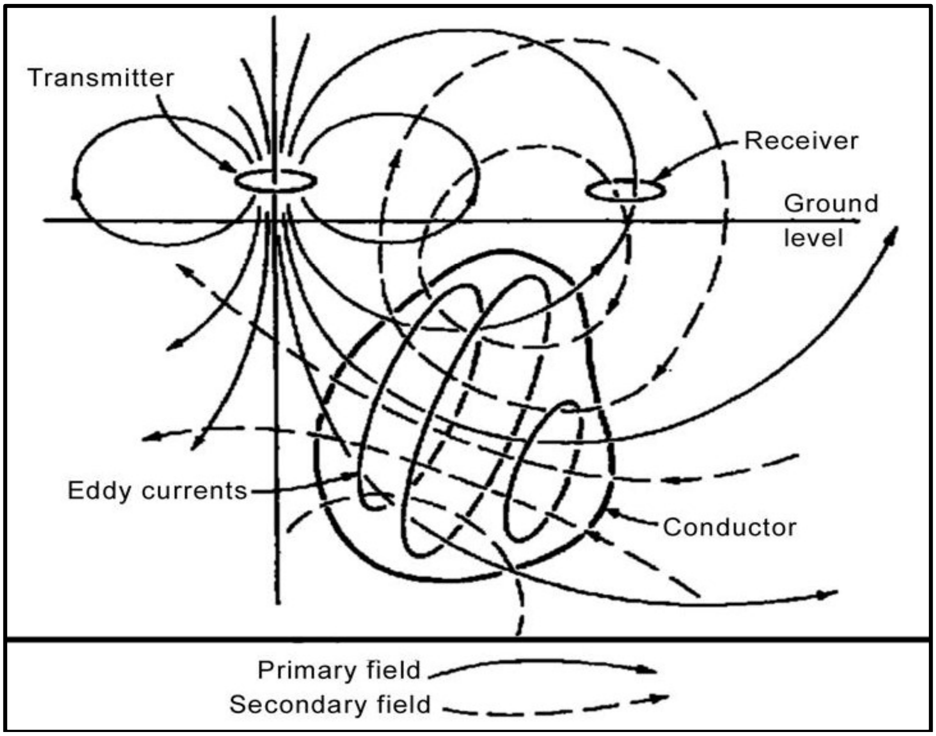


Figure 5: EM surveying Method [31]

EM profiling's fast and simple field implementation leads to a high acquisition speed, which combined with the simplicity of the data processing results in a very low cost, thus lending itself as an ideal method for initial, overall and complementary tool. For extremely rapid data acquisition over very large areas, airborne devices are also available.

Advantages of the (EM) method include the following:

(i) ability to collect data without ground contact,
(ii) rapid data collection over large areas, and;
(iii) high horizontal resolution, which enables easier anomaly identification through simpler signatures.

Disadvantages include the following:

(i) limited depth of investigation, typically no greater than 15 ft (4.6 m), for most systems,
(ii) sensitivity to aboveground and buried metallic objects, and;
(iii) instruments are subjective to interference from nearby alternating current electrical sources.

6.2 Electrical Resistivity Tomography (ERT)

(ERT) is one of the most well-established, and widely applicable geophysical techniques for detailed investigation for Measuring material electrical resistivity (inverse of electrical conductivity). (ERT) studies have similar detection capabilities to (EM) i.e. soil properties, geological structures, seepage detection, cracking, presence of metallic objects; though in much greater detail. The electromagnetic profiling is based on the measurement of induction of the primary electromagnetic field of the transmitting coil in the surrounding investigated medium.

The primary field induces a secondary field in the ground whose intensity depends on the conductivity (resistivity) of the medium surrounding the transmitting coil. Therefore, water ingress can be detected [32].

Field implementation is straightforward, but fairly time consuming and can be problematic. It involves installing large array of electrodes at a depth of up to 15cm as in Figure (6). These electrodes are connected in sequence to map a site in terms of apparent ground resistance. A pair of electrodes apply an electric current into the ground and a second pair measures the resulting electrical potential distributions as seen in Figures (7a,b). This potential, together with the current value and a constant based on the array geometry yields an apparent resistivity reading of the underlying geological layering. Measurements are taken for different 4-electrode combinations; increasing the spacing between them increases the depth of measurement but decreases resolution. Various spacing configurations are used in practice and have different advantages, so it is often useful to implement more than one such configuration.

The acquisition time is longer for this process in comparison to (EM) method, largely due to time setting up and calibrating electrode arrays. An (ERT) array can be configured to map the entirety of almost any site/ dam size, and advanced equipment is available that allow for analysis across full frequency spectrums or incorporates complimentary geophysical surveying methods such as self-potential and induced polarization.

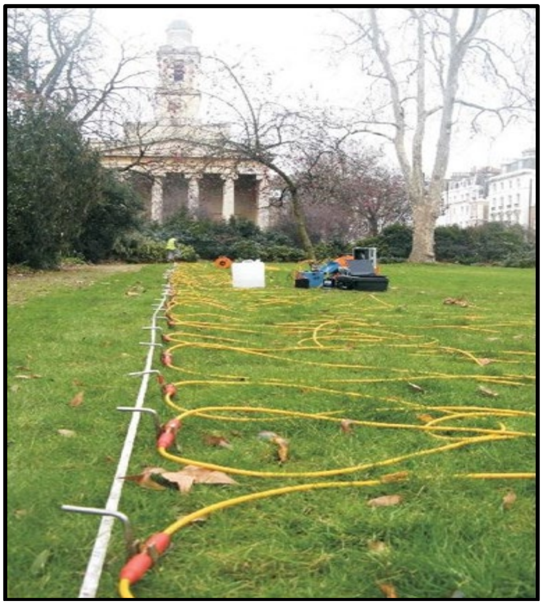


Figure 6: Typical array of electrodes for ERT [33] RSK electrical resistivity imaging.

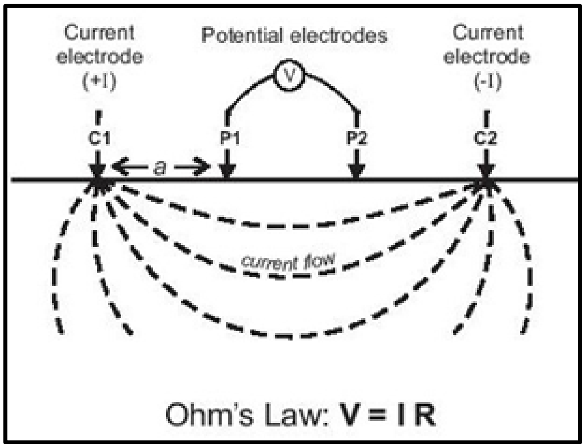


Figure 7a: Principles of ERT [33].

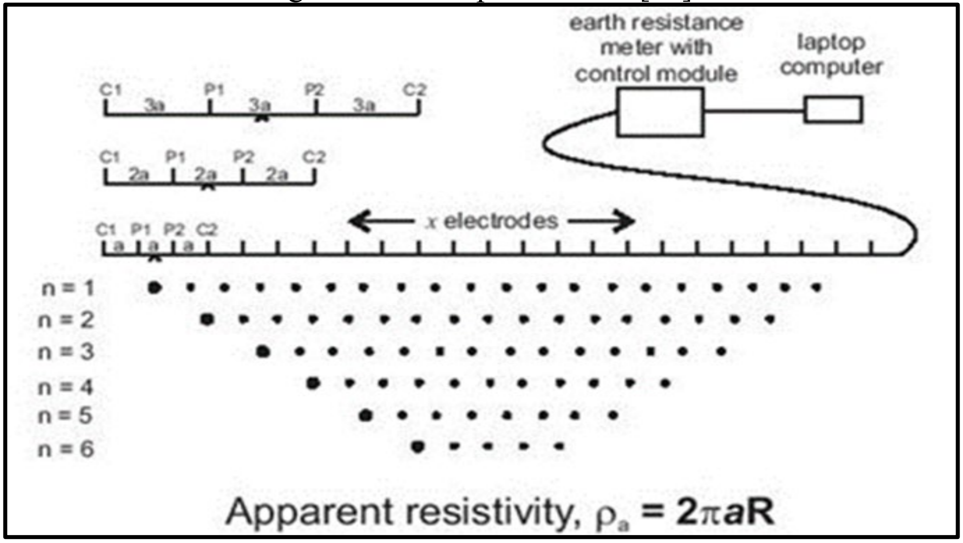
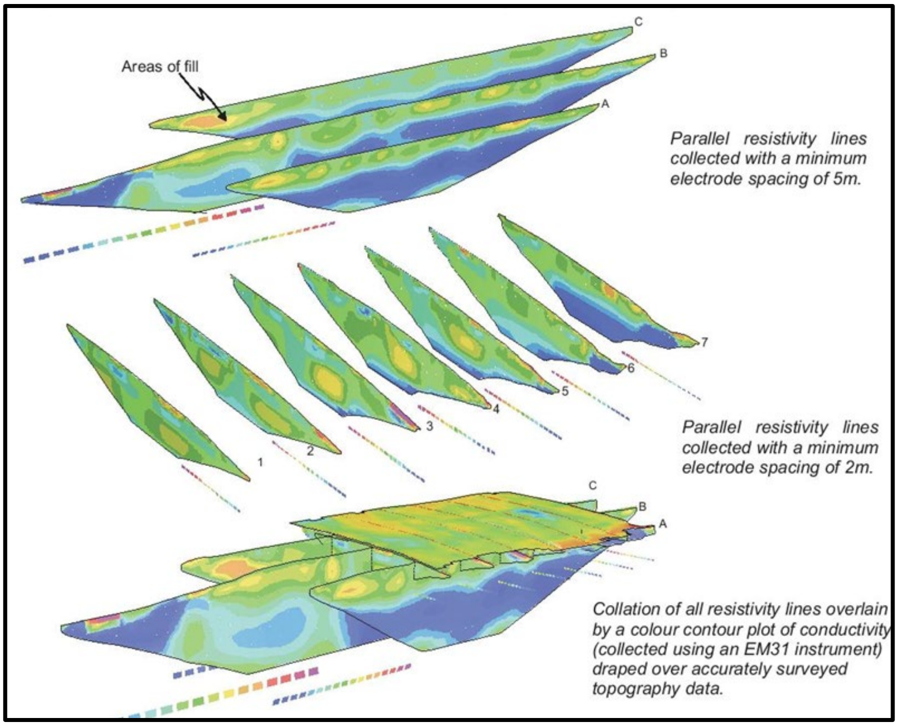


Figure 7b: Principles of ERT [33].

The raw (ERT) data is processed using a method known as inverse processing to produce (2D) models that can be transformed as pseudo 3D maps. Figure (8) presents an example of a survey over waste site. A number of resistivity lines were collected in two orientations. During processing they have been corrected to allow for the topography of the surface and have been combined in (3D) fence diagram to aid interpolation.

(ERT) is the most well-tested, widely applicable method of detailed investigations, and the procedures are clearly defined. Field work and analysis/processing should be carried out by the same team to allow accurate interpretation of the data. Much work has been carried out using (EM) and (ERT) as complementary methods. Similarly, (ERT) needs corroboration with other data to help rectify the ambiguities in inversion processing. Comparison of (ERT) method with other geophysical techniques has already been given in table (8).



Parallel resistivity lines collected with a minimum electrode spacing of 5m.

Parallel resistivity lines collected with a minimum electrode spacing of 2m.

Collation of all resistivity lines overlain by a colour contour plot of conductivity (collected using an EM31 instrument) draped over accurately surveyed topography data.

Figure 8: 3D representation of ERT [33].

6.3 Self- Potential Method

The self-potential, or spontaneous potential (SP) method is a passive technique that measures the naturally occurring potentials in the ground and it is suitable for sites under hydraulic load such as impounding dams, and dikes/levees during flood conditions i.e. transient seepage. It is the only geophysical technique that responds directly to fluid flow. Water flowing through the pore space of soil, not fully developed sinkholes and foundation cavities generates electrical current flow. This electro-kinetic phenomenon is called streaming potential and gives rise to (SP) signals that are of primary interest in dam seepage studies. Past seepage investigations have indicated a relationship between (SP) anomalies and seepage flow, with negative anomalies recorded above areas of downward or horizontal flow and positive anomalies recorded above areas with upward seepage flow [34].

Anomalous recordings could also result from numerous sources other than seepage flow, such as electrochemical activity created through oxidation reactions, groundwater recharge, and telluric currents to name few. However, surveys can be conducted with proper consideration for other sources, leading to appropriate interpretation of seepage anomalies from (SP) data [35].

The main advantages of self- potential are:

(i) Self-potential survey is conducted mainly for detecting the seepage or leakage from or through engineering structures like dams, dikes, landslide zones, canals weirs etc. (SP) Surveys have also been effectively used for mineral exploration and faults mapping [36].

(ii) It provides an estimation of the seepage intensity and not just the preferential path of flow.

(iii) It is extensively used in groundwater mapping, especially for instances where a seepage related issue has been identified but needs further analysis or conformation.

(iv) (SP) is straightforward but reasonably slow to implement; requiring only two nonpolarizing electrodes. Usually one is stationary electrode located at a base station and another roving electrode such that potentials being measured between the two.

(v) Data can be processed qualitatively very easily to produce contour maps of electric potential distributions such as that in Figure (9) [37], or quantitatively via (2D) inversion processing to furnish information about the origin of the source of potential.

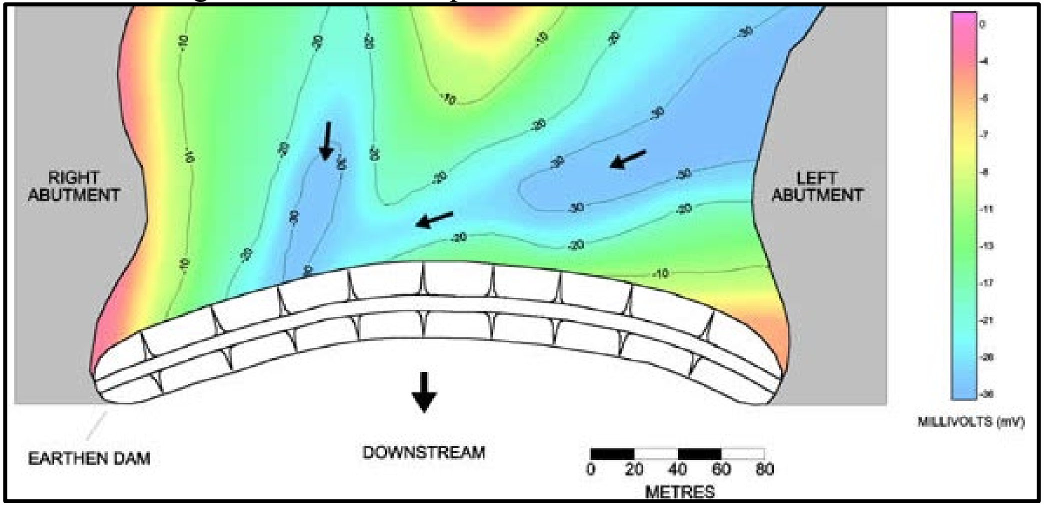


Figure 9: Color contour plot produced by SP method for a reservoir [36].

However, this method is very susceptible to noise from interfering sources of potential such as telluric currents, industrial currents, stray currents, currents associated with buried metal, and electric railway systems which can limit the scope of (SP's) use. It can also be less effective in salt water applications, restricting its applicability for coastal defense monitoring.

Normally, streaming potential surveys are generally supplemented with complementary resistivity imaging surveying for confirmation of results and obtain detailed and desired information about the seepage/leakage paths.

The basic concept of (SP) utilizes the fact that various potentials are produced in natural ground or within the subsurface altered by human actions. Natural potentials occur about dissimilar materials, near varying concentrations of electrolytic solutions, and due to the flow of fluids. Electro-kinetic, or streaming, potential is due to the flow of a fluid with certain electrical properties passing through a pipe or porous medium or cavity with different electrical properties as shown in Figure (10), which explains the basic principle of (SP) measuring technique.

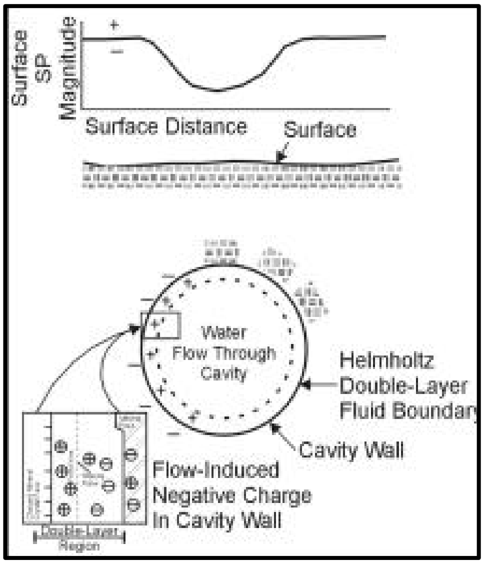

Surface SP Magnitude
+
−
Surface Distance
Surface
Water Flow Through Cavity
Helmholtz Double-Layer Fluid Boundar
Cavity Wall
Flow-Induced Negative Charge In Cavity Wall
Double-Layer Region

Figure 10: Schematic of flow-induced negative streaming potentials [37] [38].

Self- Potential is measured by determining the voltage across a pair of non-polarizing electrodes using a high impedance voltmeter. This inexpensive and deceptively simple data acquisition procedure requires special care and attention to reliably interpret and correct for sources of electrical noise that can mask the signal of interest. Typically, (SP) anomalies on the order of tenths of millivolts are associated with seepage anomalies of interest. Distance between electrodes usually ranges between several meters to tens of meters, depending on the resolution required. Guidance on site deployment, data acquisition and interpretation is given in manufacturers literature and manuals [39].

For dam safety applications, (SP) and resistivity methods generally appear to hold more promise than seismic methods as nonintrusive techniques applied at the surface of a dam. However, in specific settings, cross- holes seismic techniques could prove indispensable. Although the understanding of (SP) as applied to embankment dams has come a long way in recent years, more research is required before this technique can enter into standard practice and be applied with confidence on a routine basis. It is imperative that dam owners and practiccuting, and interpretation of the results. Geophysical data interpretation is not unique and should be constrained by incorporating various data sets. Thus, strong cooperation between the geophysicists and engineers recognize the limitations and the care required in planning, exesicist and engineers is essential to improve the interpretation and usefulness of the results [34] and [40].

6.4 Ground Penetrating Radar (GPR)

Ground penetrating radar (GPR) profiling is a technique that has come into regular use for various geophysical studies in recent years due to its simple and fast data acquisition coupled with a good level of detail/resolution [41].

The (GPR) method is based on the study of the propagation of high frequency electromagnetic waves in the ground; from a few tens of MHz to several GHz. These waves are generated in the form of temporal pulses at one point on the surface by an emitting antenna. The waves propagate within the soil at a speed typical for the terrain. When the waves meet different materials, they partially reflect back to the surface in accordance with the Snell-Descartes law, which is used to describe the relationship between the angles of incidence and refraction when referring to light or other waves passing through a boundary between two different isotropic media, such as water, glass, or air [42].

Wave's characteristics are measured by another antenna and are subsequently analyzed to deduce the properties of the soil. (GPR) helps to detect dielectric contrasts existing in the material through which the electromagnetic waves propagate. The waves are reflected in the medium, because of the dielectric contrast between materials, associated with variations in lithology, texture, porosity, material density, and particularly water content. The greater the dielectric permittivity contrast, the higher the reflection coefficient [43].

In GPR profiling, electromagnetic waves are transmitted into soil as seen in Figure (11). When these waves reach a boundary between two materials of different dielectric permittivity, they are partially reflected and refracted. The amplitude and travel time of the reflected wave is related to the dielectric permittivity ε0 of the new material the wave interacted with [44]. This method has been successfully used to detect anomalies very close to the surface both on dike slopes and on crests of embankment dams [45] and [46].

(GPR) was used as a monitoring tool in Sweden for the embankment dams at Suorva, Akkats, Grundsjön, Stenkullafors, and Näs dams by using both crosshole electromagnetic wave tomography and (GPR) reflection measurements from the ground surface. In these dams, incidents of internal erosion, increased leakage were an evident problem. Differential settlement of the soil subsequently caused sink holes to develop on the dam crest. Because of the self-healing effects of the granular sand filter zones inside the dam body these dams did not fail [47].

Ground-penetrating radar is analogous to the seismic reflection technique, except that radar (microwaves) are utilized rather than acoustic waves. Radar waves generated at the surface are reflected from subsurface boundaries separating materials of contrasting conductivity and dielectric properties and are returned to the surface. Like seismic reflection, the greater the contrast, the greater the amplitude of the reflection. Data are displayed in time in a manner similar to seismic reflection data. The velocity at which the radar waves travel through a medium is related to the dielectric constant and knowledge of this parameter can allow conversion of a time section to a depth section. Radar waves are transmitted and received by a control unit operating a transducer/antenna that is pulled along the ground. A time section is simultaneously displayed on a chart recorder, allowing the images to be examined during data acquisition. Adjustments in recording parameters can be made, as necessary. Effective depth of penetration ranges from about one foot in moist clay to 50 feet or more in dry granular sediments and rocks, and is governed by ground electrical properties. Also, higher frequency signals are attenuated faster than lower frequencies. Several antennas are available so that wave lengths can be matched to target dimensions for optimum imaging [48].

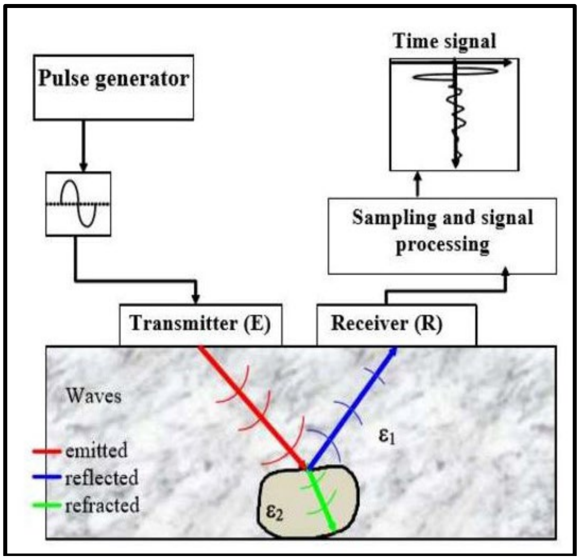


Figure 11: Flow diagram of GPR operation Principle after Royet 2013[44].

A representation of field implementation of the (GDR) method is illustrated as in Figure (12) [49].

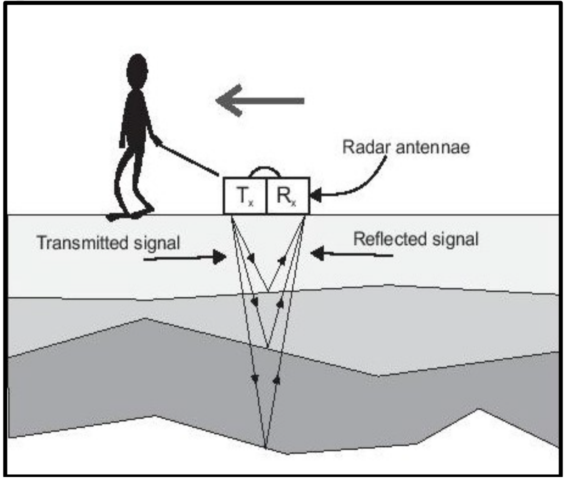


Figure 12: Field Implementation of GPR [49]

Previous seepage investigations have demonstrated the ability of (GPR) to provide useful information. Advantages include good spatial resolution and high acquisition speed. However, (GPR's) primary disadvantage is its extreme sensitivity to site conditions. Areas with high clay or water content within the shallow subsurface can attenuate the (GPR) signal, making it virtually useless.

Studies can be straightforward to carry out with a variety of devices operated by one surveyor, some handheld some towed along the ground. Although resolution can be quite high, processing/interpretation is very difficult, partially due to the sheer volume of data produced.

The depth of investigation is not very deep, restricting (GPR's) value for use on higher assets such as reservoir impounding dam, but, among the versatile uses of (GPR) are, bedrock profiling, ground water exploration, bathymetry, stratigraphy mapping and sedimentation in addition to sinkholes, and tunnel detection [50].

6.5 Seismic Methods

Although there are various types of seismic geophysical surveys that can be carried out separately or together, the basic principle of all seismic methods is the controlled generation of elastic waves by a seismic source in order to obtain an image of the subsurface. Detection here is focused on the mechanical properties of material, such as shear strength and internal structures and voids.

Seismic reflection and seismic refraction methods are reasonably easy to implement in the field, often using a transmission device as simple as a hammer, and an array of geophone receivers.

This is a very slow process which is one of the main limitations. Multi-channel analysis of surface waves (MASW) however, has a higher output rate than the two previous seismic methods but its data interpretation requires high level of expertise, so this trade-off must be considered. Seismic techniques are regularly used for deeper subsurface investigations, meaning measurements can easily be taken down to the foundations of any asset here if no other limiting factor is present. An added benefit of (MASW) is its ability to utilize vibrations caused by external noise such as traffic in an urban environment as the source of seismic waves.

These methods can be used effectively used in investigation of dams issues and discovering anomalies at an early time.

The full range of available seismic techniques include the following;

(i) **Seismic Refraction:**

Typically, acoustic pulses are generated at predetermined source locations (S) along the length of the refraction seismic profile. The travel times of acoustic energy that has been critically refracted at horizons of interest (L1) is recorded at predetermined receiver locations (R1, R2, etc.). The recorded travel time information is used to generate a velocity – structure profile of the shallow subsurface along the length of the refraction profile. If external constraints are available, the velocity–structure profile can be transformed into a geologic model, Figure (9).

(ii) **Seismic Reflection:**

This method is similar to the seismic refraction method except that the travel times and amplitudes of reflected acoustic energy are recorded at the receiver locations (R1, R2, etc.). The recorded travel time- amplitude information is used to generate a reflection seismic profile. These data can be transformed in similar way as the seismic refraction method into a velocity – structure profile, and if external constraints are available, the velocity – structure profile can be transformed into a geologic model, Figure (10).

(iii) Multichannel analyses of surface waves (MASW)

Surface (Rayleigh) wave energy, generated using a nearby acoustic source, is recorded at predetermined receiver locations (R1, R2, etc.). A dispersion curve (phase velocity versus frequency), generated from the acquired field data, is inverted and used to generate a (1-D) shear wave velocity profile which is generally tied to the physical center of the receiver array. If additional (MASW) data sets are acquired at adjacent locations, (2-D) or (3-D) shear-wave velocity models can be created. If external constraints are available, the shear wave velocity models can be transformed into geologic models, Figure (11).

(iv) Refraction Microtremor (ReMi)

Surface (Rayleigh) wave energy, generated using a passive (background) acoustic source, is recorded at predetermined receiver locations (R1, R2, etc.). A dispersion curve (phase velocity vs. frequency), generated from the acquired field data, is inverted and used to generate a (1-D) shear wave velocity profile which is generally "tied" to the physical center of the receiver array. If additional (ReMi) data sets are acquired at adjacent locations, (2-D) or (3-D) shear-wave velocity models can be created. If external constraints are available, these shear wave velocity models can be transformed into geologic models, Figure (12). This method is capable to generate a detailed vertical shear-wave velocity profile to depths of up to 100 meters. Prior to 1999, seismic shear wave profiles were obtained using shear wave refraction, seismic cone penetrometer, or downhole/crosshole techniques. The (ReMi) technique provides a non-invasive way of obtaining a vertical profile of the shear wave and, unlike borehole methods, this technique provides a shear wave and, unlike borehole methods, this technique provides a shear wave sample of a greater volume of material, thereby allowing a more representative shear wave velocity column that is averaged over the length of the seismic array. (https://www.spectrum-geophysics.com/refraction-microtremor/)

(v) Cross-Hole Seismic Tomography:

Typically, high-frequency acoustic pulses are generated at predetermined source locations (S) in the source borehole (SB). The amplitude and arrival time of direct arrivals is recorded at predetermined receiver locations in the receiver borehole (RB). The recorded travel time–amplitude data are statistically analyzed and used to generate a velocity- attenuation cross-sectional model of the area between the source and receiver boreholes. If external constraints are available, the velocity- attenuation profile can be transformed into a geologic model Figure (13).

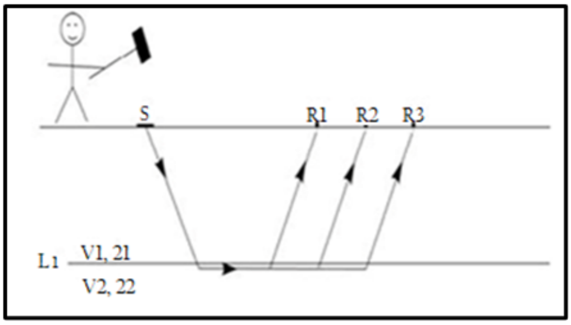


Figure 9 Seismic Refraction [51]

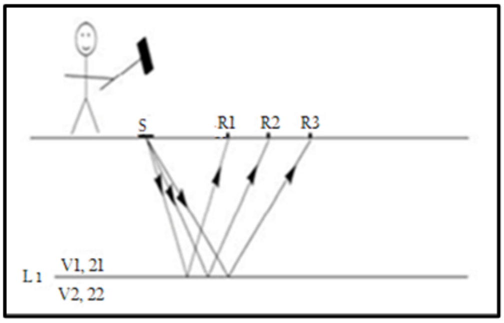


Figure 10: Seismic Reflection [51]

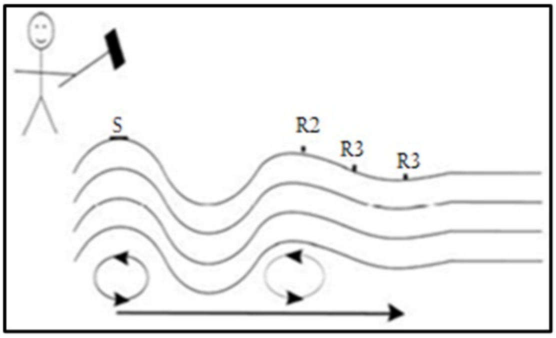


Figure 11: Multi Channel Analysis of Surface Waves (MASW) [51]

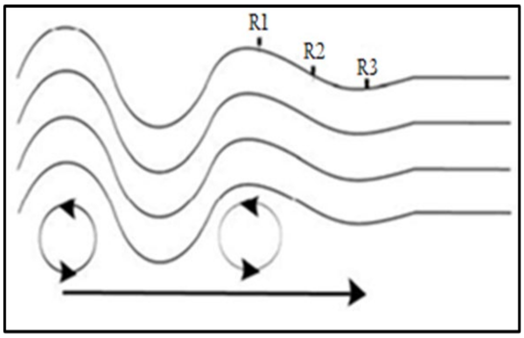


Figure 12: Refraction Microtremor (ReMi) [51]

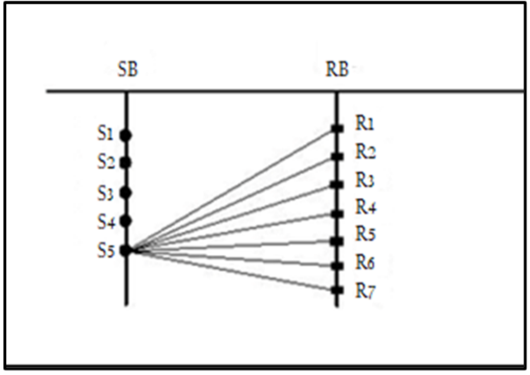


Figure 13: Cross-Hole Seismic Tomography [51]

In the seismic refraction method, the disturbance is created by shot, hammer, weight drop, or some other comparable method for putting impulsive energy into the ground. Detectors laid out at regular intervals, measure the first arrival of the energy and its time. The data are plotted in time - distance graphs from which the velocities of the different layers, and their depths can be calculated [52]. In the seismic reflection shots are fired, in turn, at each of the geophone positions and active geophones are progressively added ahead of the shots, and taken up from behind the shots, in a roll- along fashion. At each subsurface boundary, across which the elastic and density parameters differ, a percentage of the energy in the wave is reflected back to the surface where it is recorded. If a particular boundary is horizontal, the reflection point will be half way between the shot and any given geophone. Reflecting boundaries are mapped out as the system rolls along. Placing each reflection point in its proper place in the subsurface requires intense and complicated processing of the collected information [53].

Summary of commonly used geophysical seismic methods for geotechnical investigations is given in table (9) (After Anderson) [51].

Table 9: Summary of commonly used geophysical seismic methods for geotechnical investigations [51] (After Anderson 2006).

Geophysical Method	Measured Parameter (s)	Physical Propert or Properties	Physical Property Mode (Geotechnical Application)	Typical Site Model (Geotechnical Applications)
Shallow Seismic refraction	Travel time of refracted seismic energy (p- or s wave)	Acoustic velocity (function of elastic moduli and density)	Acoustic velocity – depth model often with interpreted layer boundaries	Geologic profile
Shallow Seismic reflection	Travel time and amplitudes of reflected seismic energy (p- or s- wave)	Density and acoustic velocity (acoustic velocity is a function of elastic moduli and density)	Acoustic velocity- depth model often with interpreted layer boundaries	Geologic profile
Multi-Channel Analysis of Surface Waves (MASW)	Travel time of surface waves energy generated using an active source (e.g., sledge hammer	Acoustic velocity (function of elastic moduli and density)	Acoustic (shear- wave) velocity- depth model often with interpreted layer boundaries	Geologic profile
Refraction Micrometer (ReMi)	Travel times of passive surface wave energy	Acoustic velocity (function of elastic moduli and density)	Acoustic (shear- wave) velocity- depth model often with interpreted layer boundaries	Geologic profile
Cross- hole seismic tomography	Travel times and amplitudes of seismic energy (p- or s wave)	Density and acoustic velocity (acoustic velocity is a function of elastic moduli and density)	Model depicting spatial variations in acoustic velocity	Geologic Profile

Potential geotechnical applications of these seismic methods as related to dams' issues are summarized in table (10) which is based on table (2) (after Anderson 2006) [54].

Table 10: Seismic methods as related to dams and related issues; after Anderson 2006 [54].

Application	Refr.	Refl.	ReMi	MASW	Seis Tomo.
Dam sites integrity studies	M	M	M	M	M
Foundations integrity studies	M	-	M	M	X
Landslide site evaluation	M	-	X	X	M
Mapping bedrock topography (< 30ft depth)	M	-	M	M	-
Mapping bedrock topography (> 30ft depth)	X	M	M	M	M
Identifying near surface karstic sinkholes, brecciated, and otherwise disrupted ground	M	M	-	-	-

Note: M= Major application; X= Minor Application

6.6 Temperature Methods

Temperature measurements are a well-established form of monitoring that make use of the seasonal variations that act on a waterbody. These measurements make use of natural seasonal temperature variations to locate areas of preferential seepage. Temperature in the saturated part of an embankment dam primarily is governed by the temperature of the water seeping from the reservoir. However, the air temperature from above and geothermal heat flow from below also influence temperature distribution in the dam. Geothermal heat flow is relatively constant, but air and reservoir temperatures vary seasonally and create temperature "waves" that penetrate the dam. Conductive air temperature variations typically penetrate about 10 meters below the dam surface along the crest and downstream slopes. Upstream, reservoir water exhibits seasonal fluctuations that are influenced by stream inflows and mixing. Stratification often exists in large reservoirs, and variations up to about 20 degrees Celsius (C) can occur in the upper tens of meters of the reservoir, with little seasonal fluctuation at depth. Figure (14) shows the effect of seasonal fluctuations on a vertical temperature profile measured in a deep reservoir in northern British Columbia.

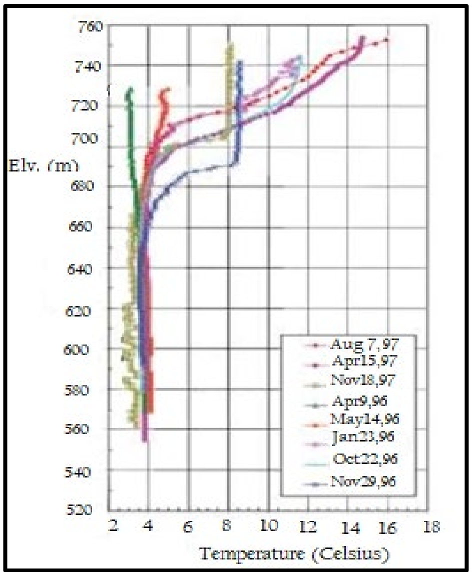


Figure 14: Vertical temperature profiles measured in the reservoir impounded by a dam in British Colombia [55].

The two major techniques of data gathering for water impounding projects involve either attaching discrete thermistors to the face of the dam, or inserting rows of temperature sensors into the body of the dam via small boreholes as shown in Figure 15 [56].

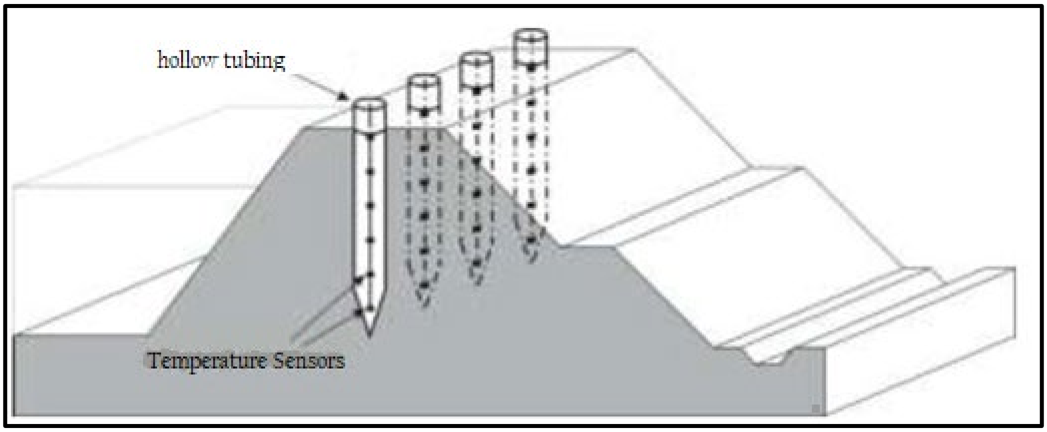


Figure 15: Schematic sketch of the ground temperature sounding technique [56].

The latter method, known as temperature sounding, is particularly useful because it can reliably detect seepage using water temperature as a tracer. The boreholes needed are small enough that they can be dug using hand tools, and the sensors can be left in for long-term monitoring. Indeed, repeated measurements can allow for determination of seepage velocities rather than just seepage identification, similar to, though not as precise as self-potential. Neither method can be carried out on a dry asset however one advantage of temperature sounding is that water temperature anomalies remain detectable for a good period of time after a flood event which can be about 10 times as long as the event itself [56]. This makes the technique more applicable to dikes and levees than (SP) in spite of that in general it is still not the preferred method. Temperature measurements are most effective when the natural gradient in the water temperature is maximized, so studies are best carried out at the height of summer or winter.

Temperature gradients between the surface water and the ground, which are due to the yearly cyclic variations, are used to detected temperature anomalies along and below a water retaining system, Figure (16). Red line indicates the natural temperature distribution. The blue line indicates a leakage zone in depth between about 8 and 12m [57].

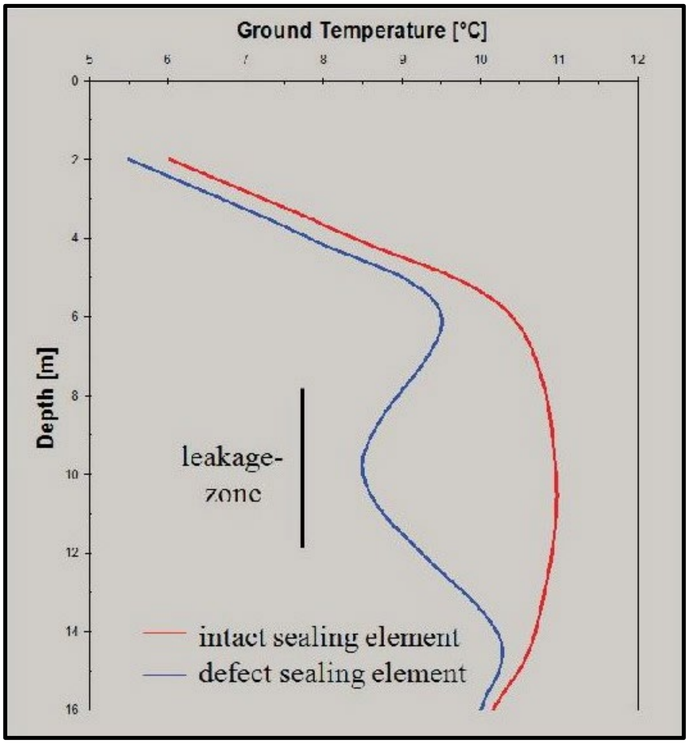


Figure 16: Temperature anomaly in ground temperature depth profile [57].

Kappelmeyer, established in Germany in 1955, was the first who used ground temperature measurements in1957 for the detection of leakages within and under the embankment dams of the Dortmund-Ems Channel in Germany. At that time Kappelmeyer was using conventional temperature sounding methods. However, until the mid-1990's distributed temperature sensing technology was developed and became economically feasible in particular for the application of temperature measurements in large dams. The German GTC Kappelmeyer company founded by Kappelmeyer in 1992 installed over 80 leakage detection monitoring systems since 1996 worldwide. Until 2017, more than 500 km of dams and embankments have been investigated. Using this ground temperature sounding technique, temperatures in earthfill dams can be measured to depths in excess of 30m, allowing seepage zones to be located. Many leaks and zones of increased permeability in tested dams have been detected, and accurately located. In addition, zones of increased flow within the foundation of dams have been identified. The technique can also be used to undertake quality assurance investigations, following new construction or remedial works. Further development of the temperature sounding technique has allowed parameters to be determined; these include, permeability and groundwater flow velocity [58]

This method is often favored by engineers/surveyors, probably due to its likeness in implementation to a more traditional geotechnical study. Discrete data can be interpolated to produce a tomography of the site, Sounding is unaffected by the presence of metal, or interfering potential sources that can often disrupt the geo-electric techniques. It can be employed in almost all materials; so long as there is a temperature contrast between it and the water, and it is unperturbed by vibrations that could be present, especially in urban environments. Temperature sounding is quite time-consuming and comparatively expensive for the level of detail it can garner from a site, but its reliability and resilience to external interference lends itself as a useful technique for particular situations [56]. Advancements in this field have led to higher sensitivities in detection using fibre optic temperature measurement devices. Figure (17) is an example of the contour plots that can be created from data processing, and demonstrates an area of leakage being detected as high temperature anomalies under a slurry trench wall.

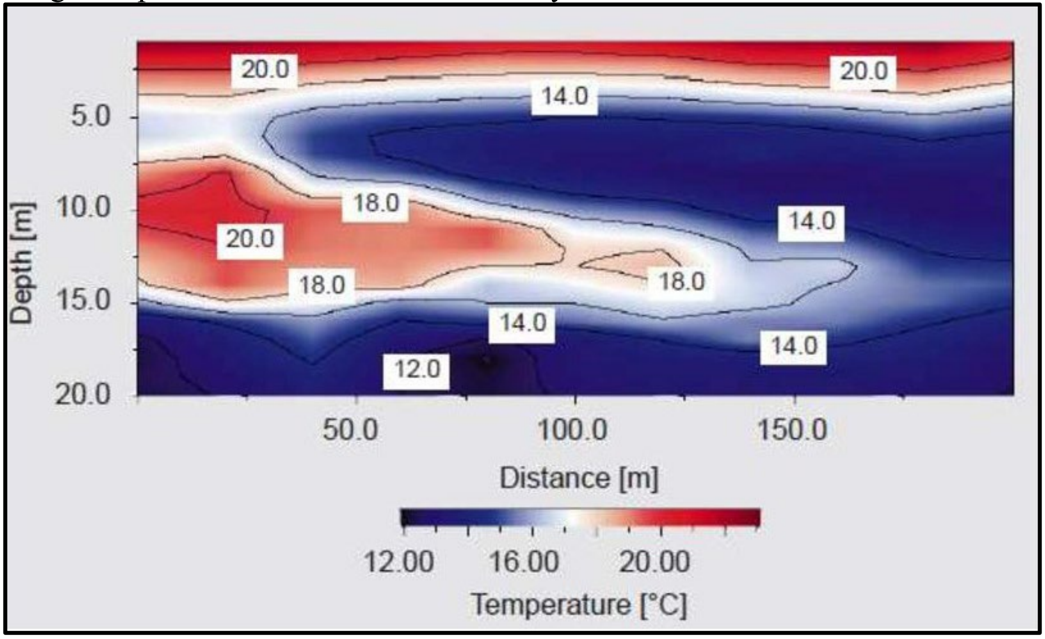


Figure 17: Contour Plot data using sounding method [58] and [59]

6.7 Other Methods

There are other geophysical methods which have been used on limited scale in dikes and earthfill dams' observations. Of these are, the microgravity, the magnetic profiling and the radio- magnetotelluric techniques.

(i) Microgravity Method

The basic theory on which microgravity technique is based is that; different subsurface materials have different bulk densities. Surveys using this technique, seeks to detect areas of contrasting or anomalous density by collecting surface measurements of the Earth's gravitational field. The method uses highly sensitive instrumentation to take measurements of variations in gravitational acceleration due to the presence of high or low density material, which would register as "negative gravity anomalies".

Gravity anomalies arising from natural or man- made subsurface features; such as voids and cavities are superimposed on much larger variations due to height latitude and regional geological variations. In order to isolate the subtle signals of interest, careful data acquisition and processing are required. The diagram in Figure (18) represents an idealized condition of such an occurrence [60].

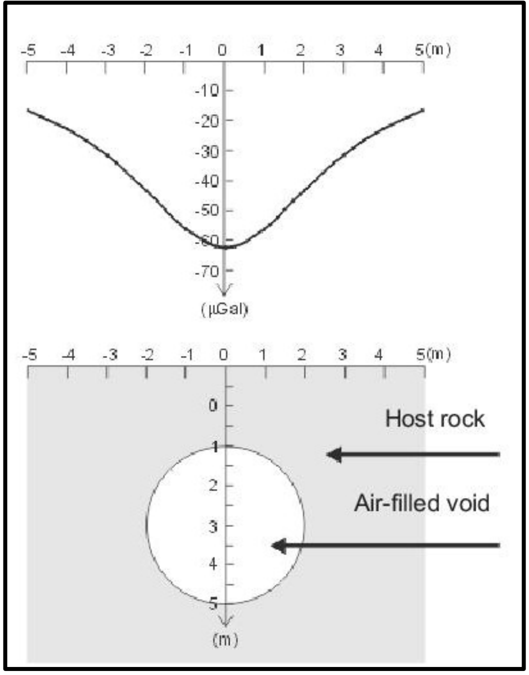


Figure 18: Idealized representing voids by microgravity survey [60]

Data collection for this process is extremely time consuming and delicate as the apparatus is extremely sensitive, requiring rigorous methodology, and data cannot be gathered continuously. The extent of measurement is confined to limited area so there is risk of missing anomalies. Moreover, this type of surveys lacks the ability to detect contact surfaces between layers of contrasting materials or condition such as compaction, permeability or cracks, buried channels in foundations, and it is severely susceptible to limiting factors in the environment [61]. These factors result in microgravity being one of the most expensive geophysical techniques.

(ii) Magnetic Profiling

Overall magnetic profiling technique of site surveys is a quick and cost effective way as a first step to be investigated further by (ERT). This significantly reduces costs and survey time, as (ERT) does not need to be implemented across the whole dam or dike. This method, however, has very narrow range of detection capabilities and suffers from limited outputs and severe limitations.

The method has no ability to detect vertical structure such as layers, depth to foundation, water table, nor the capability to delineate contact surfaces between layers of contrasting materials or identify conditions of compaction and permeability. It cannot discover anomalies such as, cracks, animal burrows, subsidence, karst cavities and buried channels in foundation, seepage areas and potential erosion and piping.

Foe materials properties and condition identification it cannot estimate soil geotechnical parameters such as; porosity, consolidation, bulk density, permeability, nor seepage flow velocity estimation [61]. Typical instrument used for this method is shown in Figure (19) [62].

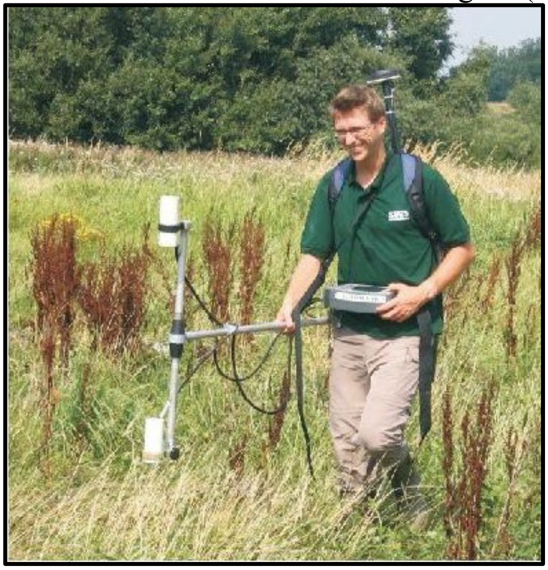


Figure 19: Typical Magnetic Profiling Instrument [62]

(iii) Radio-magnetotelluric Method

Radio-magnetotellurics (RMT) is a low- frequency far field electromagnetic method. The radio transmitters used are the source of such far fields, and the electromagnetic waves received at the measuring points at the crest or foot of the dike are plane waves. These waves propagate through the surface of the soil and if conducting or resistant anomaly is present the induced field detected at the surface is modified to indicate such anomalies. The response from an anomaly in the ground depends on the orientation of its major axis compared with the direction of primary field [63].

7. Summary Points, Discussion, and Concluding Remarks

History of geophysics and its applications to potential dam site investigations and observing behavior of existing dams have gone through many stages of development since the late 1920s until now. At the present there are the possibility of using quite large number of methods and modern techniques to investigate such phenomena as; seepage paths, seepage intensity, presence of permeability anomalies such as cracks, cavities and sinkholes, and materials layering in dam bodies or their foundations. Geophysical methods, however, are still considered as complementary to geotechnical methods of borehole drilling, test pits excavation, in addition to visual and instrumental observations and surveys. In comparison, geophysical methods can give quick answers to developing problems in non-intrusive, much faster and cheaper ways than geotechnical methods.

Geophysical methods applications use various techniques and sub- techniques aiming at measuring various soil and rock properties such as shear modulus, bulk density, porosity as well as lithology patterns, depth to bed rock and fault location, and they can infer from the observed changes the development of safety problems that can threaten the use and integrity of dams like; increasing seepage flow, development of new cracks or enlargement of old ones, development of cavities and sinkholes. It has been concluded from many study cases that different geophysical methods can supplement each other in characterizing and identifying subsurface problems. Ideally geophysical investigations may include more than one method at the same time, and then the results have been tied into existing borehole data to provide the best characterization results. Geological data from boreholes give a context to geophysical data interpretation by defining what geological features are possible or likely.

Each of the geophysical methods used today has some aspect which makes it more suitable to follow a particular issue or can presents certain limitations. Electromagnetic (EM) profiling method for example measures conductivity of soil and rock by measuring changes in the electromagnetic field created by inducing an electric potential field in different materials. Electrical Resistivity Tomography (ERT), however, measures directly the (resistivity) which is the inverse of (conductivity) of materials by creating an electric field and mark the resistivity of the materials in this field. Self-Potential (SP) technique, on the other hand, measures changes in the various types of naturally existing electric potentials in the ground, such as the electro- kinetic potential due to the flow of a fluid with certain electrical properties within a cavity or porous medium. (SP) method is, therefore, more suited to transient flow phenomena measurements than the (EM) and (ERT) methods which are more adapted to measuring static conditions.

Progress in modern geophysical methods coupled with developments of new instruments has led in the last few decades into new and modern techniques, such as the use of the Ground Penetrating Radar (GPR) and Seismic Methods (SM) and temperature sounding.

The (GPR) method is based on the study of the propagation of high frequency electromagnetic waves into the ground and measuring changes in the partially reflected waves. It can detect changing properties and conditions related to the dielectric contrasts existing in the materials through which the electromagnetic waves propagate. This method is limited in its application to detect anomalies which are close to the surface of slopes and crests of dikes and earthfill dams. The more laborious Seismic Methods (SM), however, lend themselves to variety of sub-techniques which are all based on the controlled generation of elastic seismic waves by seismic source and obtain an image of the subsurface. Detection here is focused on the changing mechanical properties of soil materials such as shear strength and presence of internal structures and voids, and in this they can present indirect evidence of newly developing seepage paths and cavities development.

The variety of sub- seismic techniques used today include; Seismic Refraction and Seismic Reflection techniques, the Multichannel Analysis of surface waves (MASW) [64], in addition to the use of Refraction manometer (ReMi) and the Cross- Holes Seismic Tomography.

The difference between Seismic Refraction and Seismic Reflection techniques can be illustrated in Figure (20) [65]. These methods, however, are based on creating energy pulses in the ground and make measurements which depending on the technique used can record travel time or travel time amplitudes of the received pulses, or record the surface energy waves (Rayleigh Waves) as is the case in the (MASW), and (ReMi) technique. The pulses are recorded and presented as (Phase Velocity vs. Frequency) dispersion curve in (1D) shear wave velocity profile that may be transformed into (2D) and (3D) models. In such cases, if external constraints are present, then these models can be changed to geologic models.

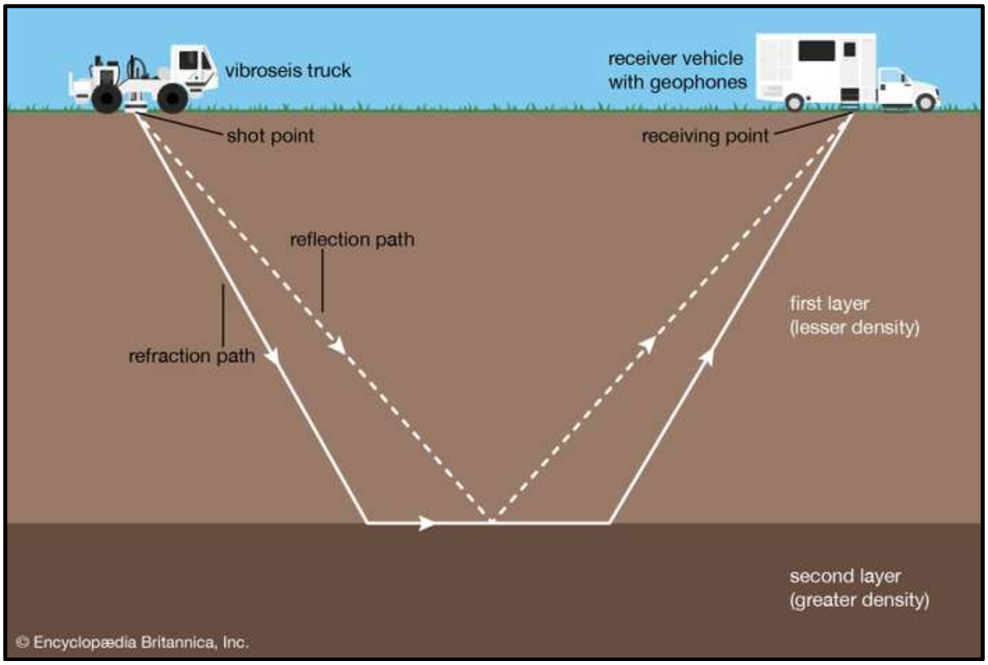

vibroseis truck
receiver vehicle with geophones
shot point
receiving point
reflection path
refraction path
first layer (lesser density)
second layer (greater density)
© Encyclopædia Britannica, Inc.

Figure 20 : Illustration of Seismic Refraction and Seismic Reflection techniques [65].

Temperature measurements have also found their way of application in dams and dikes geophysical monitoring where reliable detection of seepage is done by using water temperature as a tracer. These measurements make use of natural seasonal temperature variations to locate areas of preferential seepage and this is performed by either by attaching discrete thermistors to dam face or inserting rows of temperature sensors into the dam body via small boreholes dug into the dam. Discrete data can be interpolated to produce a tomography of the site. This technique, however, is quite time consuming and comparatively expensive but its other traits lend it useful for particular situations.

Other modern techniques which have been used on limited scale in dams and dikes observation include; i) the Microgravity Measurement method based on collecting surface recordings of the Earth gravitational field variations that result from the changes in materials density and give negative gravity anomalies in case of the presence of low density materials or voids and cavities; ii) Magnetic Profiling method which maps variations in the magnetic field of the Earth that are attributable to changes of structure or magnetic susceptibility in certain near-surface rocks, and; and iii) the Radio Magnetotelluric method which relay on emitting low frequency far field waves and receiving them at measuring points at the crest or foot of a dam after detecting any conduction or resistance anomaly within the dam body.

From the vast majority of case histories on the application of geophysical methods in observation of existing dams very little is found in applying these methods to concrete dams. One such case, however, is that of Marathon Dam (Greece), which is a typical old concrete gravity dam, (54) m high, that was constructed in (1926) and is located about (30) km north of Athens. The dam was tested for appraisal of the geotechnical properties of the dam body and its geological setting, and for tracing possible leakages. Moreover, this was meant to detect also possible degraded areas that are potentially liable to water infiltration, and to evaluate the dynamic properties of the subsurface materials. The investigations were extended in the dam interior in order to evaluate the quality of the concrete. This survey was necessary since the dam was hit by a strong earthquake (Ms= 5.9) in 1999. Geophysical methods of seismic and electrical tomography, as well as ground penetrating radar (GPR), were utilized to test the dam aiming to properly evaluating its overall safety. The results indicated a few suspicious parts in the dam that warranted close inspection by specialists [66].

Multiple cases of applying various geophysical methods have been applied, to embankment dams other than those listed in tables (1,2,3). The (ERT) technique was used in Zaria Dam (Nigeria) to check for suspected seepage channels in this earthen dam. The variations of the water content in the dam body was suspected to be of an anomalous seepage beneath the subsurface. On the basis of the interpretation of the acquired data, various zones of relatively uniform resistivity values were mapped and identified. The available borehole log data correlated well with the obtained resistivity values and depths. Zones of relatively low resistivity within the bedrock were interpreted to represent potential seepage pathways. Hence, this geophysical method was successfully used to delineate and map these seepage pathways within the subsurface of this earth dam [67]. A similar case of seepage anomaly detection was discovered in Ogobomboso earthfill dam, also in (Nigeria), where Self-Potential (SP) and (ERT) were conducted and two possible seepage zones were identified within the dam embankment. The integrity of the dam embankment was fairly good but the existence of seepage zones beneath it could constitute serious threat to the safety of the dam [68]. Other similar cases may be cited from the Kaffrein Dam (Jordan) [69], Riverbanks dikes and embankments for a group of reclamation and irrigation channels located between Malalbergo and Barricella, Bologna (Italy) [70], Oba earthfill dam (Nigeria), [71], and the Røsvatn field test dam in Norway [72] and many others.

Detection of seepage within foundations of embankment dams was also performed for many earthfill dams using geophysical methods such as; the Vitineves reservoir Dam (Czech Republic) [73], Success Dam, California, (USA) [74], Beaver Dam, Arkansas (USA) [75], an unnamed homogeneous earth-fill dam located at the base of the Rocky Mountain foothills in Jefferson County, Colorado (USA) [76], Wadi Megenin Dam (Libya) [77] Krousovitis Dam (Greece) [78], Mill Creek Dam, Walla Walla, Washington (USA) [79], Coursier Lake Dam, British Colombia, Canada [80], Wolf Creek Dam, Russell County, Kentucky, (USA) [81].

In all these cases, one or combination of many geophysical techniques were used to get valuable knowledge on the behavior of the earthfill dams and dikes under consideration. In many of the cited examples, these investigations were carried out before performing further geotechnical investigations as a way to plan these additional investigations, or they were done in support of geological data already available. It may be said that such methods have proven their extreme usefulness for early detection of seepage and other anomalies related to seepage or internal erosion, and they have helped in planning repair works in much faster and cheaper ways than the conventional geotechnical investigations. In any similar case, therefore, their use is highly recommendable on condition of proper selection of the method and its proven suitability to the check problem at hand.

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Dams Safety: Review of Satellite Remote Sensing Applications to Dams and Reservoirs

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Abstract

Remote sensing is the collection and interpretation of information of an object, area or phenomenon by a recording device that is not in physical or intimate contact with the object or phenomenon under study. It generally refers to the use of satellite borne or airborne sensors to capture the spectral and spatial relations of objects and materials on Earth from the space. This is done by sensing and recording reflected or emitted electromagnetic radiation from the objects. A brief history of satellite remote sensing is given in this review but the bulk of it is devoted to the scientific satellites launched into orbit and their sensors tracking, and presenting changes in water resources fields. The used technologies and satellite systems for monitoring movements and changes include American (GNSS), (GPS), the Russian (GLONASS), Europe's European Satellite Navigation System (GALILEO), China's (COMPASS/BeiDou), the Indian (IRNSS); Japan's (QZSS) and many others. Details are presented on the present (LANDSAT), the Moderate Resolution Imaging Spectroradiometer (MODIS), as well as Synthetic Aperture Radar (SAR), and (RADARSAT), (JERS-1), and (ERS), which are developed by various countries, especially the USA. These sensors have the refined capability of providing estimates of variables, which depending on the purpose and design of the sensor, can follow critical issues related to water management problems. This review presents examples of actual studies carried out including; building databases of small dams and lakes on regional scale, derivation of volume vs. elevation and surface area vs. elevation of hundreds of reservoirs around the world, various bathymetric reservoir surveys, siltation of reservoirs and catchment areas erosion problems, monitoring of water quality changes, and above all monitoring dam

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deformation and stability problems of dams. The presented case studies cover the use of these different sensor together with the imagery used, their sources, methods of interpretation, validation and gives presentation of the end results. This review, which is only a very brief presentation of satellite remote sensing applications concludes that; in spite of the large volume of research done on this subject so far, which this review cites some of them , the expected future developments in satellite remote sensing technology coupled with advances in algorithms and models used in refining satellite imagery and validating the results will bring more accurate results and less laborious treatment work in addition to wider scope of applications.

Keywords: Remote sensing, satellites, sensors, electromagnetic radiation, GNSS, GPS, GLONASS, GALILEO, COMPASS/BeiDou, IRNSS, QZSS LANDSAT, MODIS, SAR, RADARSAT, JERS-1,ERS, Bathymetric surveys, siltation of reservoirs, erosion, water quality, dam deformation.

1. General

Remote sensing is the collection and interpretation of information of an object or an area without being in physical contact with it. It involves the use of instruments or sensors to "capture" the spectral and spatial relations of targets observable at a dis-tance from the space and analyzing and applying that information. The process in-volves the elements of (energy source, radiation and the atmosphere, interaction with the target, recording energy by the sensors, Transmission, reception, and processing, interpretation and analysis, finally the Application). So remote sensing provides a bird's eye view of much larger extent by looking down from above. The total area encompassed is considerably enlarged when looking downward from, say, a tall building or a mountain top. This increases even more from an airliner cruising above 10000 meters. From a vertical or high oblique perspective, the obtained impression of the surface below is notably different than when observing the surroundings from a point on the surface [1]. In this information era, the trend is of remote sensing is to use new technologies for automation of monitoring important engineering undertak-ings or natural hazard such as dams, reservoir water level fluctuations and landslides [2]. This is especially important when it comes to accurately monitor absolute (3D) movements of a dam for example. In many cases, dam movement is measured by means of plumb lines, inclinometers or tiltmeters, and manual geodetic surveying techniques. Automation can be done here by using advanced instruments and means, and from that an absolute displacement can be estimated and recorded. There are even now many emerging technologies like robotized total stations, ground based radar, etc. which rely on very expensive and delicate equipment to be installed on site and subject to vandalism. .

The problem is even worse when it is required to monitoring slopes, abutments and landslides. The inaccessibility of some sites makes the task a huge challenge[3].

Satellite technologies, however, can provide tools to accurately monitor movements and changes without the need of expensive equipment on the ground. One example of such technology is the Global Navigation Satellite System (GNSS) which refers to a constellation of satellites providing signals from space that transmit positioning and timing data to (GNSS) receivers. The receivers then use this data to determine location and more detail information.

At present (GNSS) include many fully operational global systems; namely the United States' Global Positioning System (GPS) which became fully operational in 1993, and the Russian Federation's Global Navigation Satellite System (GLONASS) which was completed in 1995 and rejuvenated afterwards to be one of the most expensive projects in Russia's Space program budget in 2010 [4].

Other system went into orbit later on namely Europe's European Satellite Navigation System (GALILEO) which went alive in (2016), and China's (COMPASS/BeiDou), the North Star, which its last satellite was launched on June 23rd 2020, marking the completion of the deployment of China's own global navigation system. This satel-lite, the 55th in the family of BeiDou that means "Big Dipper" in Chinese, was suc-cessfully sent into the preset orbit by a Long March-3B carrier rocket, according to the launch center [5]. Then India's Regional Navigation Satellite System (IRNSS); which was completed in 2016 [6], and Japan's Quasi-Zenith Satellite System (QZSS) which had 18 satellites in operation by 2018 to be increased to 38 by 2024 [7].

Once all these global and regional systems become fully operational, the user will have access to positioning, navigation and timing signals from more than 100 satel-lites [8].

The second technology other than the (GNSS) which has been in wide use is the (Interferometric synthetic aperture radar: InSAR). This is a geodetic technique that can identify movements of the Earth's surface. Observations of surface movement made using InSAR can be used to detect, measure, and monitor crustal changes associated with geophysical processes such as tectonic activity, earthquakes, landslides and volcanic eruptions. When combined with ground-based geodetic monitoring, such as Global Navigation Satellite Systems, InSAR can identify surface. The technique can potentially measure millimetre-scale changes in deformation over spans of days to years. in particular monitoring of subsidence and structural stability [9]. Also, it's very important techniges for generating Digital Elevation Model: DEM by using the Interferometric Phenomenon.

GNSS and InSAR are two different approaches that can provide tools to accurately monitor movements and changes without the need of expensive equipment on the ground, they complement each other to get a robust and complete solution. GNSS can provide fully automated motion monitoring in (3D) and real-time to a millimetre level accuracy for a discrete number of points by means of affordable (GNSS) receivers on the ground. InSAR on the contrary, provides infrequent, near vertical movements of a large number of points over a wide area also to millimetre-level accuracy without any equipment on the ground. Together, a comprehensive reservoir motion monitoring can be achieved. The radically two different approach provide robustness to the system to better comprehend the behavior of the dam- reservoir system [10].

Remote sensing has become now a major technological and scientific tool used to monitor planetary surfaces and atmospheres. In fact, from a dollar-based measuring stick, the expenditures on observing Earth and other planets has since the earliest days of the space program now exceeded $150 billion. Much of this money has been directed towards practical applications, largely focused on environmental and natural resource management.

2. Historical Development of Remote Sensing

Remote sensing began in the 1840s as balloonists took pictures of the ground using the newly invented photo-cameras [11]. In the early 20th century remote sensing images were captured using kites and even by pigeons carrying small light weight cameras attached to these birds, and photos were automatically taken for military aerial reconnaissance using a timing mechanism. In 1906 professional photographer George Lawrence used a string of kites to raise a 49 pound camera 1000 feet in the air to capture the devastation of the earthquake in San Francisco. The steel kite line carried an electric current to remotely trigger the shutter. The famous photograph "San Francisco in Ruins" was taken 6 weeks after the earthquake and subsequent fires in San Francisco Figure (1)[12].

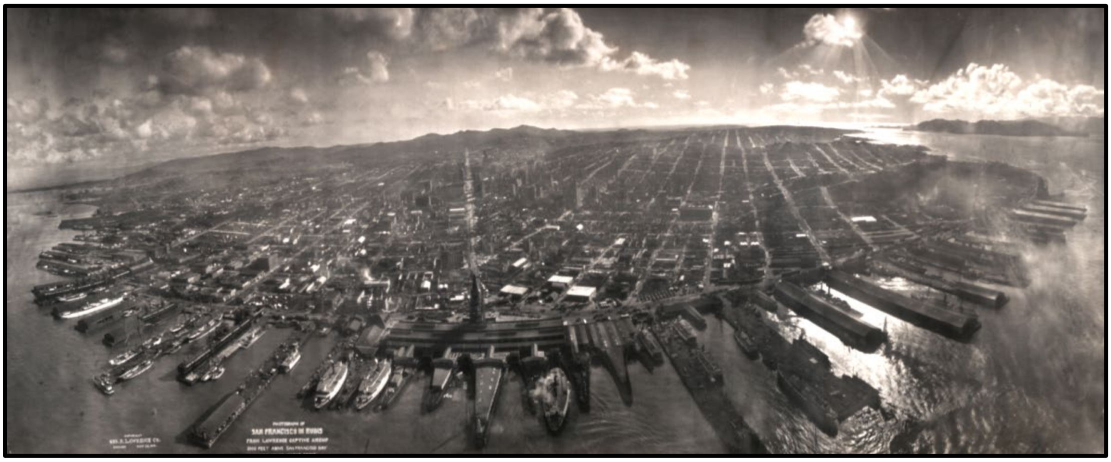


Figure 1:"San Francisco in Ruins," by George Lawrence 1906 [12].

Aerial photography became a valuable reconnaissance tool during the first world war and came fully into its own during the second world war [12][13]. It was used for espionage in the (U2) flights program during the Cold War era, which was started in 1956 and came to an end on 1st May 1960 when a (U2) plane flown by Francis Gary Powers was shot down by (SAM) missile over Russia.

Use of aerial photography to produce maps in much faster and cheaper way compared to traditional ground surveys was used in the United States for farm programs beginning in the 1930s with the passing of the Agricultural Adjustment Act. The Agricultural Adjustment Administration (AAA) began its aerial photography program in 1937 and by 1941 the (AAA) has flown and acquired aerial photographs of more than 90% of the agricultural land in the US.

The development of satellite based remote sensing began with the (Space Race) by launching the first world satellite (Sputnik 1) in 1957 by the Soviet Union, followed by the United States in 1960 with the successful launch of (Explorer1). In 1972 (Landsat 1), the first earth resource satellite was launched by the US. The main goal of the (Landsat) program was to collect data from the Earth through remote sensing techniques. Landsat 1 was originally named (Earth Resources Technology Satellite 1). The Landsat program has continued for 45 years with (Landsat 8) launched in 2013 and with (Landsat 9) expected to be in orbit in mid-2021.

Since the launch of (Sputnik 1) thousands of satellites have been launched. There are great number of commercial and governments satellites in operation today, many of which are used for remote sensing applications. There are currently over 3,600 satellite orbiting the Earth, but only approximately 1400 are operational. Of these satellites, there is well over 100 which are earth observing satellites that carry variety of different sensors to measure and capture data about the Earth. These satellites are often launched by governments to monitor Earth's resources, but private commercial companies are becoming increasingly active in launching earth observing satellites as well.

Seasat was the first satellite designed for remote sensing of the Earth's oceans with synthetic aperture radar (SAR) which was launched on 27th June 1978 [14]. On the 35th anniversary of (Seasat's) launch, the Alaska Satellite Facility (USA) released newly digitized Seasat synthetic aperture radar (SAR) imagery [15].
Until this release, (Seasat SAR) data were archived on magnetic tapes, and images processed from the tapes were available only as optical images of film strips or scanned digital images. Neither the tapes nor the films allow the quantitative analysis possible with the new digital archives. With the great improvements in computer-based processing, especially now, the ability of personal computers to handle large amounts of remote sensing data has made satellite and manned platform observations accessible to universities, resources-responsible agencies, small environmental companies, and even individuals.

Application of remote sensing in the field of water resources has become nowadays one of the principle tools in following and solving problems of water quality, runoff and hydrological modeling, flood management, watershed management, drought management, irrigation command area management, snow cover and glaciers, groundwater studies [16][17] [18] in addition to solving problems of transboundary reservoir systems monitoring and dam safety monitoring [19].The need to develop monitoring systems that can map changing land use, search for and protect natural resources, and track interactions among the biosphere, atmosphere, hydrosphere, and geosphere has become a paramount concern to managers, politicians, and the general citizenry in both developed and developing nations.

3. General Principles of Satellite Remote Sensing

There two types of remote sensing imaging systems:

i) Passive remote sensors; which gather radiation that is emitted or reflected by the object or surrounding areas. Reflected sunlight is the most common source of radiation measured by passive sensors and microwaves emissions is another one.

Examples of passive remote sensors include film photography, infrared charge-coupled devices, and radiometers that whereby the radiant energy is converted to bio-geophysical quantities such as temperature, precipitation, soil moisture, chlorophyll etc. These sensors can be: :

First: the passive microwave sensor (TMI) designed to provide quantitative rainfall information over a wide swath under the Tropical Rainfall Measuring Mission (TRMM) satellite by carefully measuring the minute amounts of microwave energy emitted by the Earth and its atmosphere. (TMI) is able to quantify the water vapor, the cloud water, and the rainfall intensity in the atmosphere. It is a relatively small instrument that consumes little power. This combined with the wide swath and the good quantitative information regarding rainfall make (TMI) the "workhorse" of the rain-measuring package on Tropical Rainfall Measuring Mission [20], and; Second: the Moderate Resolution Imaging Spectroradiometer (MODIS) that was launched into orbit by (NASA) in 1999 on board the Terra satellite, and in 2002 on board the Aqua (EOS PM) satellite. They are designed to provide measurements in large-scale global dynamics including changes in Earth's cloud cover, radiation budget and processes occurring in the oceans, on land, and in the lower atmosphere, and they have been used in monitoring of large dams reservoirs storage.

ii) Active remote sensors: Active sensors create their own electromagnetic energy that is transmitted from the sensor towards the targets, interacts with the targets producing a backscatter of energy and is recorded by the remote sensor's receivers.

RADAR, LiDAR and LADAR are examples of active remote sensing where the time delay between emission and return is measured, establishing the location, speed and direction of an object [21]. In both LiDAR and LADAR, remote sensing is achieved by illuminating the target with laser light and measuring the reflection. Differences in laser return times and wavelengths can then be used to make digital (3-D) representations of the target [22]. LiDAR stands for Light Detection and Ranging and (LADAR)is abbreviated from; Laser Detection and Ranging. They are both acronyms that represent one type of remote sensing technology that can determine the distance between a sensor and an object. Coupled with the known location of the sensor, the range information can be used to produce a highly detailed 3-dimensional map of the object [23]. The Principles of passive microwave sensor and active remote sensors performances are illustrated in Figure (2).

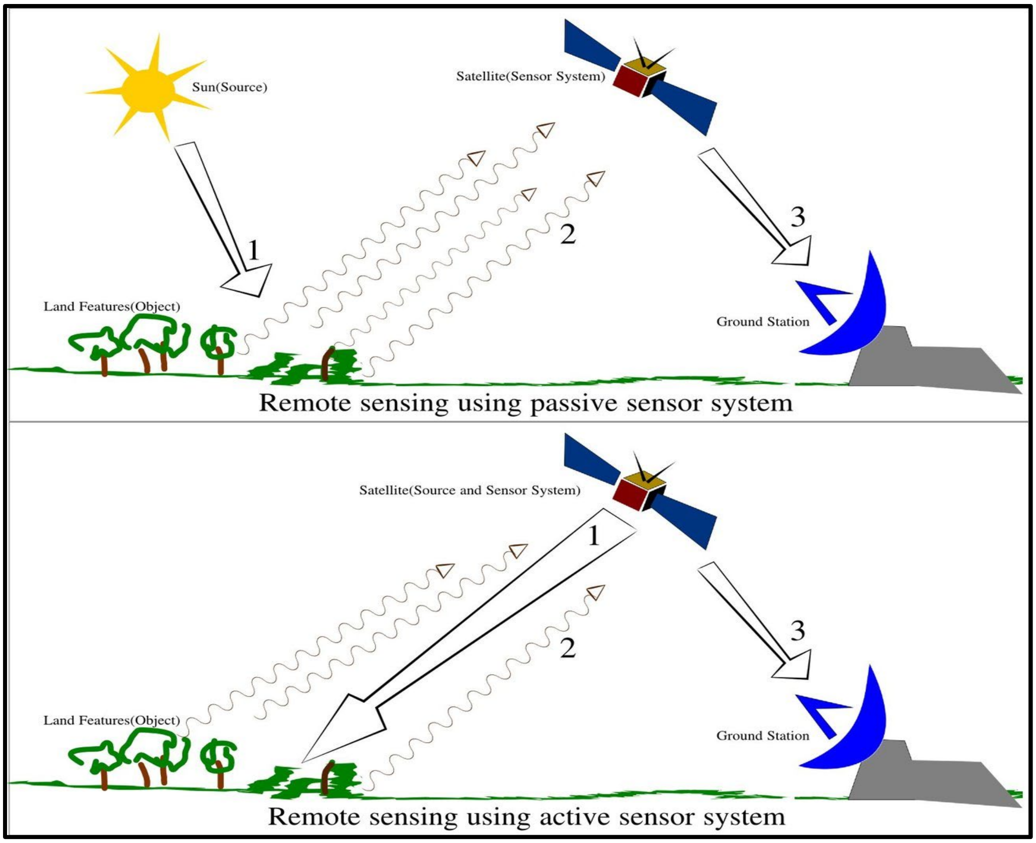

Sun(Source)
Satellite(Sensor System)
1
2
3
Land Features(Object)
Ground Station
Remote sensing using passive sensor system
Satellite(Source and Sensor System)
1
2
3
Land Features(Object)
Ground Station
Remote sensing using active sensor system

Figure 2: Passive and active sensors systems working principles [24]

In the passive remote sensing systems, the sensor itself is composed of several radiation detectors. Assembling a set of radiation detectors with different wavelength band sensitivity and adding the necessary hardware will result in a multispectral sensor. The recording of image generating data by multispectral sensors is often performed by multispectral scanning. A set of sequential scan lines will eventually build an image, Figure (3) [25].

The smallest element to be detected by a sensor is called the geometric resolution of the system. The detector will measure the average reflection in a selected wavelength band. The result will be recorded as an image element on the scan line called a pixel. the pixel size that is cited, e.g. 80 m pixel size means that the smallest image element records the average reflection from an area of 80 by 80 m i.e. 6400 m2.

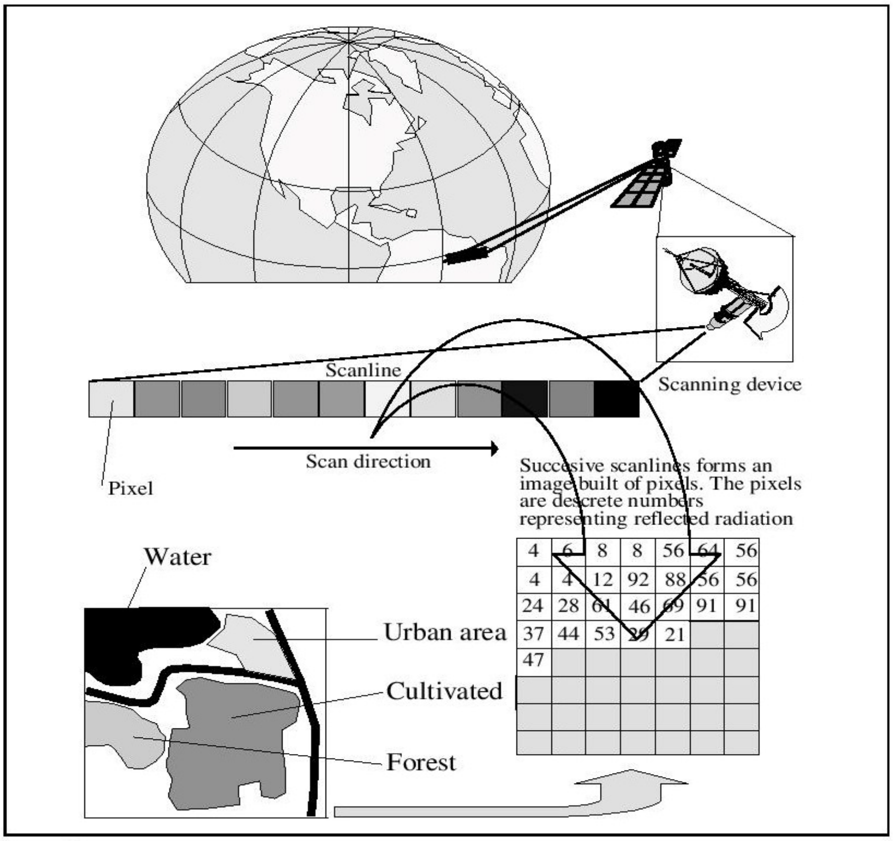


Figure 3. The chain of satellite data acquisition. The scanning device records data, pixel by pixel, along a scan line. Assembling the scan lines gives an image. The radiation is recorded in grey levels, most satellite sensors have an 8-bit resolution that permits 256 levels [25]

In cases of detecting, for example, a dry water course passing through an irrigated agriculture area. The reflectance from the river will differ very much from the surroundings since it lacks vegetation and has a bed of coarse sand and gravels with a high reflectance. The radiation detector will take an average reading from every pixel in the area. Pixels containing only irrigated fields will have a low reflection and appear dark on an image. Although the river is only 20 m wide it will significantly influence the average reading of the reflectance from all pixels it passes through. This will render these pixels brighter than the surroundings and on an image a series of pixels, following the river bed will get significantly different radiation readings, making the river appearing as a line on the image [25][26][27]. To summarize a satellite (sensor) image is built by pixels which are the smallest element in the image. A number of pixels are forming a line (scan line), recorded perpendicular to the flight direction of the sensor platform. When arranged properly using dedicated software the lines form an image.

The basic information on satellite remote sensing given in the preceding pages are meant to guide the unspecialized reader in following the various applications in: First: fields of civil engineering projects in the stages of:

i- Reconnaissance.

ii- Preliminary Planning / Feasibility
iii-Design
iv- construction

v- Post construction/maintenance.

.

The potential role of remote sensing at each stage will vary from project to project, but the following sections indicate some of the possible uses of the techniques [28], and;
Second: water resources management where such applications have already been used successfully in observing flooding conditions and recording water level fluctuation in rivers and reservoirs, following reservoirs surface area variations, estimation of reservoirs storage and reservoirs siltation in addition to ascertain current dams and landslides stability conditions.

4. Dams and Reservoir Site Selection using Satellite Remote Sensing

Dams and reservoirs site selection may be done in various ways, but most of the traditional methods are costly and time consuming. Advances in remote space technologies, however, can be utilized to make optimum use of time and cost. Satellite Remote Sensing provides imaging technology for updating of available maps and information in real time frame as well as collecting data of land use and Landover. A well-selected site may give the optimum benefits with minimum problems related to its performance and its impacts on the environment. In many recent cases these methods have been supported by use of the Geographical information System (GIS) which is computer based system that handles the attribute data as well as spatial data since geographical information is an important characteristic. Use of (GIS) in many such studies is considered as a decision support system regarding optimal site selection. Remote sensing methods may provide basic data required for the (GIS) to build geodatabase and make decisions by using the appropriate available software.

As a demonstration of using such methods it is worth referring to one case study describing the process of selecting potential dams/ reservoirs for the water supply of Karachi in Pakistan which was published in 2010 [29].

The first step was to determine site selection criteria which were to be considered as constraints in this procedure. Identification and justification for each of them was given by:

i- Sites should not be in the vicinity of densely populated areas due to safety considerations,
ii- Sites must have water accumulating topography to form reservoirs,
iii- General slope of the area shall be gentle and not exceeding 11° to avoid slope failure for environmental considerations,
ii- v- Having enough water source (run off and rainfall) for satisfying the projected demand.

The required layers of data necessary for the (GIS) use were obtained by using combination of both traditional methods and by utilizing acquired remote sensing data. In this way it was possible to create and convert one information layer for each of the constraints data sets mentioned above by using (ArcGIS) software [30]. Information layers for constraints (i) to (iii) were produced from two satellite data sources; the first source was (SPOT-5), which is one of the earth observation satellites (Passive System) in the program developed by the French space agency in the 1970s in association with (SSTC) from Belgium and (SNSB) from Sweden [31]. The second source was Shuttle Radar Topographic Mission (SRTM) dated 2004. (STRM) is a (NASA) mission initiated in 2000 to obtain elevation data for most parts of the world, and it is the current dataset of choice for digital elevation model (DEM) data since it has a fairly high resolution (1 arc-second, or around 30 meters), and it has near-global coverage (from 56°S to 60°N), which is available in the public domain [32]. Finally, the rainfall data needed for (iv) and required for the study were obtained from available (1989- 2000) local records. Summary of the criteria, the required data, and their sources is shown in Table (1).

Table 1: Site selection criteria, required data and their sources

Criteria		Data	Source
i		Settlements	Satellite images (Spot 5)
ii		Elevation/ Topography	Satellite images (STRM)
iii		Rainfall	Precipitation records
iv		Drainage network	Satellite images (STRM)/ Satellite images (Spot 5)

Although this methodology makes the decision making process more objective, there is still an element of subjectivity which originates from the planner own experience and judgment. In this case selection of the more promising sites was done by allocation weights to each of the criteria and screening the result in a procedure described in the study. This procedure allows flexibility to be incorporated as varying degrees of importance are assigned to each criterion based on the planner's experience. The final decision was left to the relevant department which is the final decision maker. It is recognized that social, political and budgetary constraints could produce an alternative other than the best alternative obtained by the planner.

Our conclusion is that; the planning process of any development project of an engineering nature, such as dams, communication lines and live lines can be a complex process involving many disciples based on requirements dictated by natural elements, social rules and environmental factors. Space remote sensing can help as a tool to provide basic data of natural and environmental factors, which otherwise have to be obtained by costly and time consuming traditional methods. The great number of satellites and the numerous sensors onboard available today present good opportunities in planning, but decision making remains as an unchallenged human activity.

5. Dams and Reservoirs Management utilizing Satellite Remote Sensing Applications

Space remote sensing has found its application in USA for many of the operational issues of dams and their reservoirs since the late 1980s. Initial work was done in1987 over a small number of targets using data from (SESAT) [33]. (SEASAT) was the first Earth-orbiting satellite that was managed by (NASA) for remote sensing of the Earth's oceans and had on board the first spaceborne synthetic-aperture radar (SAR). This satellite was launched on 27 June 1978 but it stopped working on the10th of October 1978. Observation was followed after this by the extraction of decimal time series from (TOPEX) mission[34]. (TOPEX/Poseidon) was a joint satellite mission between (NASA), the U.S. space agency, and (CNES) the French space agency, to map ocean surface topography; Launched on August 10th,1992.

Estimating water storage in large reservoirs and lakes is attributed to the advent of radar altimetry. The most commonly used spaceborne radar altimeters during the past period were; (GEOSAT), (Topex/Poseidon or T/P), (ERS-1), (ERS-2), (GFO), (ENVISAT), (JASON-1) and (JASON-2).These sensors are out of service now and are replaced by other more modern sensors such as (LANDSAT), the MODIS, as well as SAR, and such as (RADARSAT), (JERS-1), and (ERS). All these sensors have the refined capability of estimating water storage and storage variation in lakes and reservoirs, in addition to measurements of both surface water area and bathymetry.

The primary advantage of (LANDSAT) is its high resolution of (30m) per pixel, but it has low repeat frequency and is susceptible to cloud cover contamination. For sensors with daily coverage, like (MODIS), the frequency of observations is obviously an advantage, but the resolution is relatively coarse (250 m to 500 m) per pixel.

As an example of studies on the use of such measurements for the derivation of the elevation- area variation time series one example is presented in the following text. It is taken from a comprehensive study which was carried out in 2012 on five of the largest reservoirs in the United States and 34 reservoirs in various parts of the world by utilizing available (MODIS) outputs. [35]. Water elevations and surface areas data during any overlapping period(s) from (2000 to 2010) for each of these reservoirs were used to derive their elevation-area relationships. These elevation-area relationships were then used to estimate reservoir storage volume time series. In Figure (4) the water surface elevation and surface area relationship for Fort Peck Reservoir is presented as one of the outputs of this study.

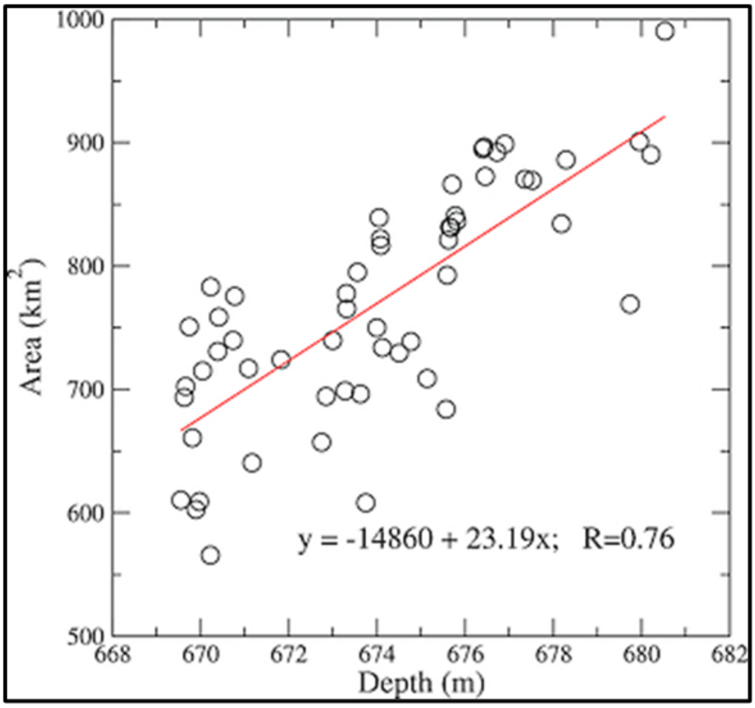


Figure 4: Water surface elevation and surface area relationship for Fort Peck Reservoir [35].

A linear regression was used to approximate the relationship between surface elevation (h) and surface area (A) i.e.,

A = f (h). For the period (1992- 2000),

when (MODIS) data were unavailable, this relationship was applied to estimate the reservoir surface area from the water elevation historical data. Similarly, during periods when altimetry data were unavailable during the (MODIS) era, the water elevation was estimated as an inverse function of the surface area function (h = f −1(A)). The storage equation:

Vo = Vc - (Ac + Ao)(hc-ho)/ 2 , was used to estimate reservoir storage,

where V c , A c , and h c represent; storage, area, and water elevation at capacity, and V o , A o , and h o are the observed storage, area, and water elevation, respectively. The values at capacity (V c) were taken mostly from the Global Reservoir and Dam (GRanD) database [36]. (GRanD) is based on multiple sources, including a variety of regional and national inventories and gazetteers, International Commission on Large Dam's World Register of Dams as well as a variety of publications, monographs, and maps.

By substituting the elevation-area relationship into above equation, the storage equation can be simplified into a single variable function, either as a function of water elevation from altimetry or as a function of surface area from (MODIS). The value of (MODIS) data was provided in filling the gaps in the existing records to derive the complete elevation- area variation time series for all of these reservoirs for the period (1992 to 2010) which then could be used to estimate reservoirs storage volume time series for the same period. In Figure (5) the time series of reservoir storage volume estimated from remote sensing for Fort Peck Reservoir is presented as an example.

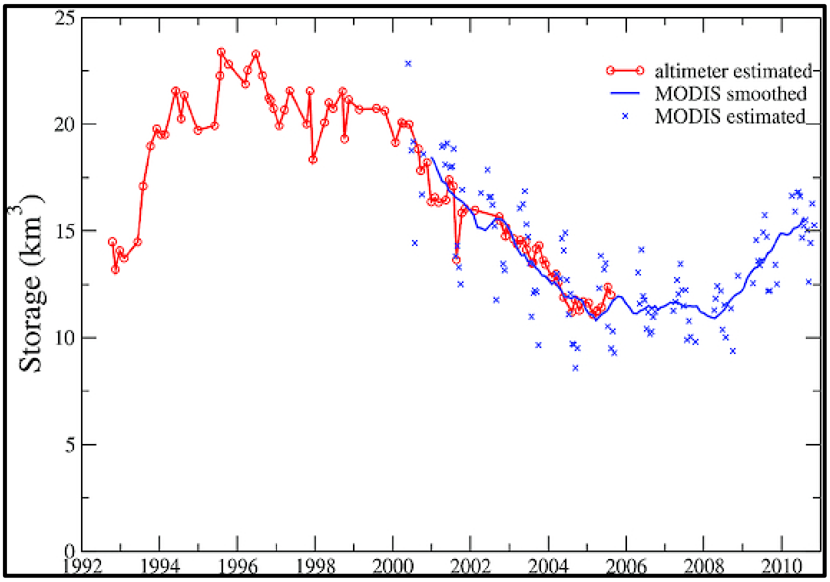


Figure 5: Time series of reservoir storage estimated from remote sensing for Fort Peck Reservoir [35].

The complete sets of the time series for water surface elevations, surface area, and storage volumes which are obtained from remote sensing for the five U.S reservoirs are given in Figure (6). This figure shows comparison of gauge observations for the reservoirs in (black), altimetry- based estimates in (red), and (MODIS) based estimates are in (green).

In the same way, the elevation-surface area relationships for each of the 34 other world reservoirs were developed, and then they were used to estimate time series of reservoir storage volumes. The reservoir storage volume results are shown in Figure (7) and the elevation-area relationships are summarized in Table (2). The total capacity of the 34 reservoirs is 1164 km3, which represents about 15% of global reservoir capacity. For 16 of these reservoirs, the estimated storage volume is available for 19 years (1992–2010) and the average record length for all reservoirs is 14.5 years. The correlation coefficient between the water elevation and surface area varies from 0.08 to 0.98, with an average value of 0.5. A high correlation usually indicates good quality for both data sets, while a low correlation can result from many conditions. These include errors from either the water elevation or surface area or both, and/or the possibility that within the range of variation the bathymetry is independent of area (i.e., vertical walls). For consistency within the time series for each reservoir, the (MODIS) estimated surface areas were used to maximize the record length, when altimetry water elevation was unavailable, but this is done only if the correlation coefficient between altimetry water elevation and MODIS surface area exceeded 0.5.

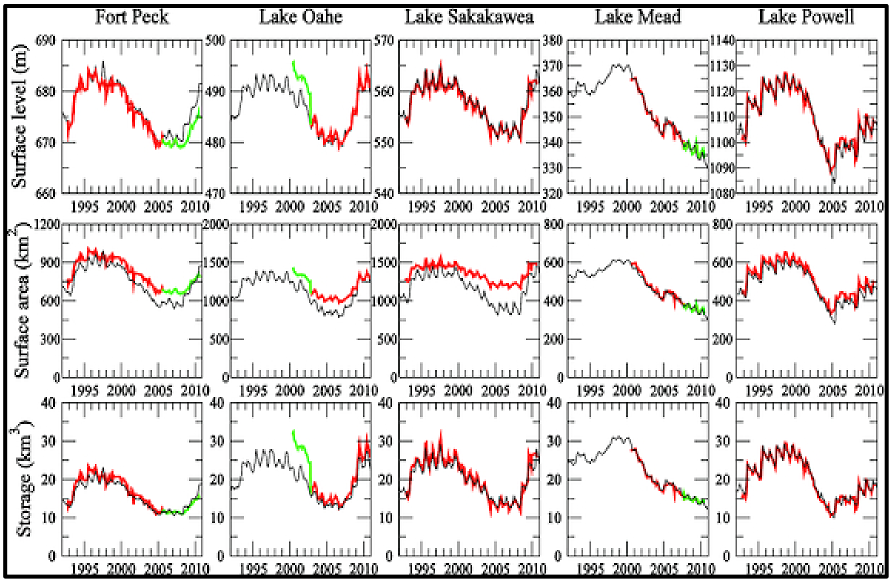


Figure 6: Comparison of gauge observations for five U.S. reservoirs; gauge observations in (black), altimetry- based estimates in (red), and (MODIS) based estimates are in (green) [35]

Three of the reservoirs given in this table are located in Iraq; Mosul Dam reservoir, Qadisiya Dam (Haditha) reservoir and the Thartar Reservoir. Of these three reser-voirs the Tharthar, reservoir and the Qadisiya Dam are selected for further com-ments, in addition to commenting on Lake Nasser on the Nile River (Egypt), and the Toktogul Dam reservoir (Kyrgyzstan) in addition to Guri Dam reservoir (Venezuela).

Lake Tharthar, the largest artificial lake in Iraq, is the first example. Its primary pur-poses are irrigation and flood control. During the study period, there were two severe droughts in the Fertile Crescent, one from 1998 to 2001 and the other from 2008 to 2010. Both wheat and barley production dropped precipitously during these drought events. The remotely sensed reservoir storage for Lake Tharthar indicates that the lowest storage during these events was about 35% of the peak value in 1993. Lake Qadisiyah (Haditha), a much smaller reservoir, built for irrigation, flood control and power generation close to Lake Tharthar, was hit relatively harder by the droughts; reservoir storage was completely depleted by the end of 2009 for a short period. Knowledge of water availability in water-sparse regions like this is crucial for man-aging irrigation water use and for planning aid.

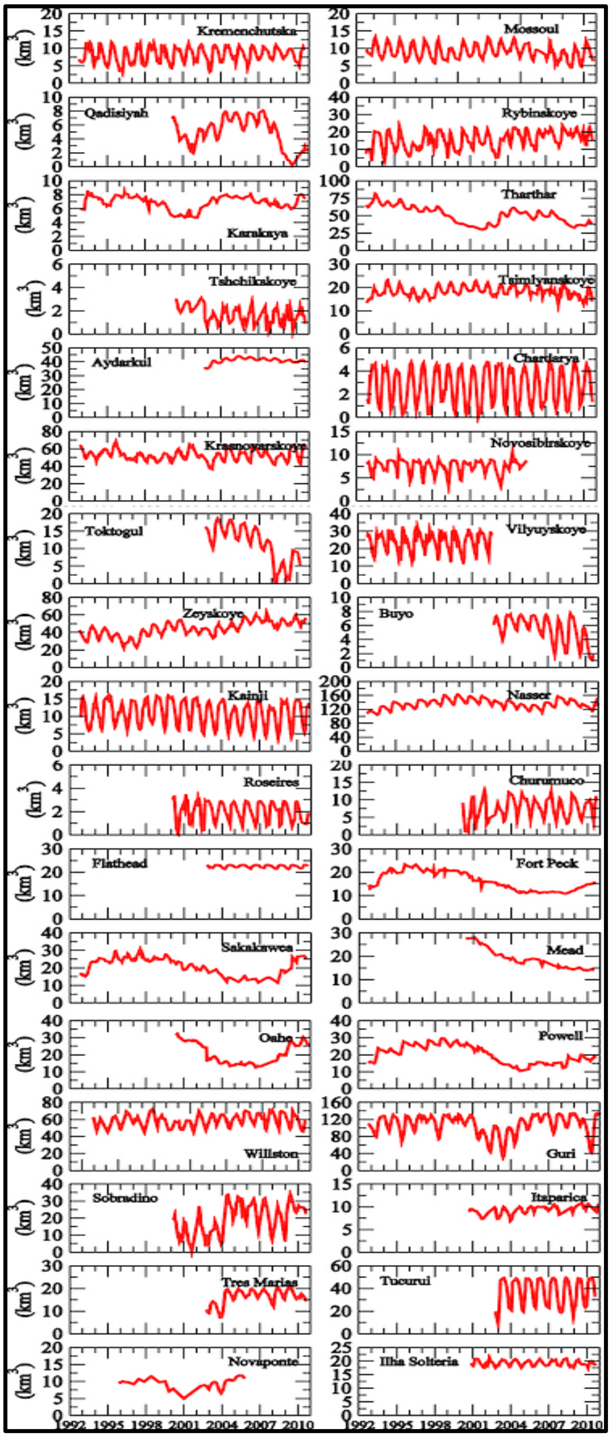
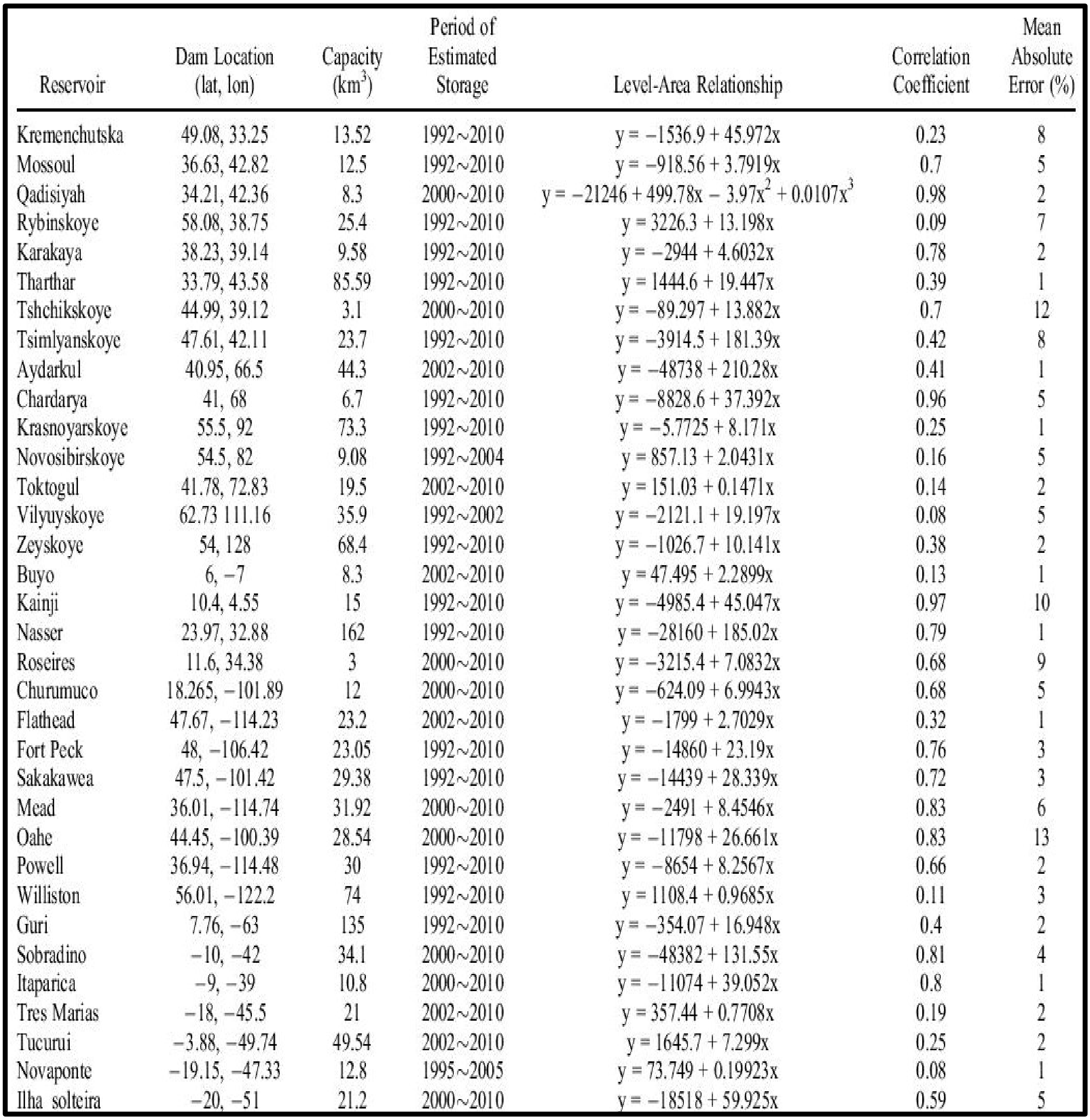


Figure 7. Global reservoir storage time series from remote sensing [35].

Table 2: Water Elevation- Area Relationships, Correlations, and Mean Absolute Errors [35]



Reservoir	Dam Location (lat, lon)	Capacity (km^3)	Period of Estimated Storage	Level-Area Relationship	Correlation Coefficient	Mean Absolute Error (%)
Kremenchutska	49.08, 33.25	13.52	1992~2010	$y = -1536.9 + 45.972x$	0.23	8
Mossoul	36.63, 42.82	12.5	1992~2010	$y = -918.56 + 3.7919x$	0.7	5
Qadisiyah	34.21, 42.36	8.3	2000~2010	$y = -21246 + 499.78x - 3.97x^2 + 0.0107x^3$	0.98	2
Rybinskoye	58.08, 38.75	25.4	1992~2010	$y = 3226.3 + 13.198x$	0.09	7
Karakaya	38.23, 39.14	9.58	1992~2010	$y = -2944 + 4.6032x$	0.78	2
Tharthar	33.79, 43.58	85.59	1992~2010	$y = 1444.6 + 19.447x$	0.39	1
Tshchikskoye	44.99, 39.12	3.1	2000~2010	$y = -89.297 + 13.882x$	0.7	12
Tsimlyanskoye	47.61, 42.11	23.7	1992~2010	$y = -3914.5 + 181.39x$	0.42	8
Aydarkul	40.95, 66.5	44.3	2002~2010	$y = -48738 + 210.28x$	0.41	1
Chardarya	41, 68	6.7	1992~2010	$y = -8828.6 + 37.392x$	0.96	5
Krasnoyarskoye	55.5, 92	73.3	1992~2010	$y = -5.7725 + 8.171x$	0.25	1
Novosibirskoye	54.5, 82	9.08	1992~2004	$y = 857.13 + 2.0431x$	0.16	5
Toktogul	41.78, 72.83	19.5	2002~2010	$y = 151.03 + 0.1471x$	0.14	2
Vilyuyskoye	62.73 111.16	35.9	1992~2002	$y = -2121.1 + 19.197x$	0.08	5
Zeyskoye	54, 128	68.4	1992~2010	$y = -1026.7 + 10.141x$	0.38	2
Buyo	6, −7	8.3	2002~2010	$y = 47.495 + 2.2899x$	0.13	1
Kainji	10.4, 4.55	15	1992~2010	$y = -4985.4 + 45.047x$	0.97	10
Nasser	23.97, 32.88	162	1992~2010	$y = -28160 + 185.02x$	0.79	1
Roseires	11.6, 34.38	3	2000~2010	$y = -3215.4 + 7.0832x$	0.68	9
Churumuco	18.265, −101.89	12	2000~2010	$y = -624.09 + 6.9943x$	0.68	5
Flathead	47.67, −114.23	23.2	2002~2010	$y = -1799 + 2.7029x$	0.32	1
Fort Peck	48, −106.42	23.05	1992~2010	$y = -14860 + 23.19x$	0.76	3
Sakakawea	47.5, −101.42	29.38	1992~2010	$y = -14439 + 28.339x$	0.72	3
Mead	36.01, −114.74	31.92	2000~2010	$y = -2491 + 8.4546x$	0.83	6
Oahe	44.45, −100.39	28.54	2000~2010	$y = -11798 + 26.661x$	0.83	13
Powell	36.94, −114.48	30	1992~2010	$y = -8654 + 8.2567x$	0.66	2
Williston	56.01, −122.2	74	1992~2010	$y = 1108.4 + 0.9685x$	0.11	3
Guri	7.76, −63	135	1992~2010	$y = -354.07 + 16.948x$	0.4	2
Sobradino	−10, −42	34.1	2000~2010	$y = -48382 + 131.55x$	0.81	4
Itaparica	−9, −39	10.8	2000~2010	$y = -11074 + 39.052x$	0.8	1
Tres Marias	−18, −45.5	21	2002~2010	$y = 357.44 + 0.7708x$	0.19	2
Tucurui	−3.88, −49.74	49.54	2002~2010	$y = 1645.7 + 7.299x$	0.25	2
Novaponte	−19.15, −47.33	12.8	1995~2005	$y = 73.749 + 0.19923x$	0.08	1
Ilha solteira	−20, −51	21.2	2000~2010	$y = -18518 + 59.925x$	0.59	5

In central Asia, the time history of storage in Toktogul Reservoir was examined. It supplies water to Kyrgyzstan's single largest hydropower plant, and it also provides irrigation water downstream. During the 2007–2008 drought, the reservoir storage was completely depleted. Storage information for the Toktogul Reservoir is also cru-cial for water management in the Naryn/Syr Darya basin, which is a major interna-tional river system in Central Asia.

With a storage capacity of 157 km3, Lake Nasser in Africa is the third largest man-made reservoir in the world by volume. Its main use is for irrigation, with hydropow-er and flood control as secondary operation purposes. During the 1990s the water elevation and storage increased due to high precipitation in the Ethiopian Highlands. For the safety of the dam, water was spilled from Lake Nasser westward into the Sa-hara Desert, forming the Toshka Lakes (beginning in 1998). From 2003 to 2007, these discharges to the Toshka Lakes were stopped, and the new lake contracted. From knowledge of storage history for Lake Nasser it shows that the rebound of stor-age in 2008–2009 should have allowed more spillover to the Sahara, suggesting pos-sible increases in irrigation diversions or releases to the Nile Delta. Guri Dam in South America is the world's third largest hydropower plant. It supplies 73% of Venezuela's electricity. Two major drought events (2001 to 2004 and 2009 to 2010) were the worst in the past 40 years in Venezuela, and the reservoir dropped to a low storage of 28% of its maximum in 2003. During the 2010 event, power ration-ing was implemented to close the electricity gap. The obtained reservoir time series shows that reservoir storage experienced a quick recovery from the last drought by the end of 2010.

The interest for securing reliable water resources in many semiarid regions of the world has increased sharply during last decades to meet the increasing population demand for food. This has led to expanding irrigated agriculture from small water sources and required the use of small reservoirs for the hitherto neglected such re-sources. The increasing number of small size dams and their wide distribution over large swaths of rural countryside has made their monitoring much more difficult. To overcome the lack of baseline data some studies have classified the extent of small reservoirs surface areas of such reservoirs from (Landsat ETM) which utilize the En-hanced Thermal Mapper sensor (ETM) born by Landsat 7 (1999) [37].

One study published in 2009 explained the use of (ENVISAT-ASAR) to monitor three small dams close to the village of Tana Natinga in upper east region of Ghana, West Africa, where water supply from these three small dams was critical for the population of three villages to satisfy irrigation demand, livestock watering and household use [38]. The area was semiarid with an annual average precipitation of not more than 986mm and the three reservoirs had shallow depths. The study used 22 (ENVISAT-ASAR) images acquired bimonthly from June to August 2006 to en-sure observing seasonal variation, the changing vegetation context, and the large var-iability of backscatter from water surfaces i.e. , through wind- induced water surface roughness. The (ENVISAT) or “Environmental Satellite” is a large inactive Earth-observing satellite which is still in orbit, Operated by the European Space Agen-cy (ESA)[39][40], and (ASAR) is the advanced Synthetic Aperture Radar sensor [41]. The overall conclusion of this research has shown that regional to basin scale inventories of small inland water bodies are readily possible with (ENVISAT-ASAR) images . In combination with regional area- volume equations, basin- wide reservoir storage volumes can be estimated, and the impact of further development can be as-sessed and monitored.

Moreover, in a research carried out in 2005 [42], the inventory of small farm reservoirs in Insiza District that forms part of Mzingwane catchment (Zimbabwe) [43] was obtained through remote sensing by acquisition of images from (LANDSAT 5, bands 5, 4 and 2). Using Red- Green- Blue channel, water bodies were depicted in colors ranging from blue to almost black and reservoirs were uniquely identified. This pro-cedure was applied for all reservoirs studied in this research work. The objectives of the study were:

i- To identify small reservoirs in terms of their numbers and spatial distribution in the study area from satellite images.

ii- To develop a methodology to estimate small reservoir capacities as a function of their remotely sensed surface areas in the basin.

The estimated number of small farm reservoirs when this study was conducted in (2005) was 1000 in Insiza District as depicted from satellite image and they were quite evenly distributed in the entire catchment as displayed in Figure (8).

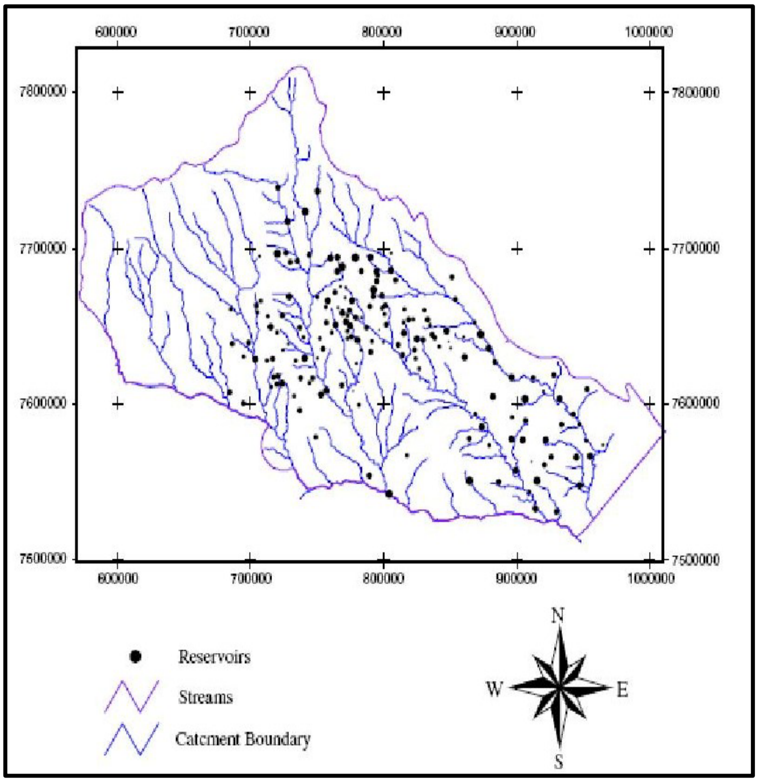


Figure 8: Spatial distribution of small reservoirs in the Mzingwane catchment [42].

This number included small reservoirs in both communal lands and resettlement are-as, which means an update baseline data of reservoirs was obtained by use of remote sensing. The method of using satellite information to estimate the number of small reservoirs and keep an update of small reservoir database proved to be useful. The difference in the number of small reservoirs' records obtained from other sources in later years maybe due to construction of small farm dams in these years by some farmers without prior approval from the water authorities. Having established the spatial distribution of small reservoirs in the study area in question, field study on 12 selected reservoirs from this inventory was carried out. The depths of water accompanied with their coordinates were measured; from which area and capacity relationships were calculated for each reservoir using geographical information system (Arc View 3.2 GIS) package with Spatial Analyst, plus Surface Areas and Ratios from elevation Grid extension [44] were used to automate surface area calculations and to provide surface area statistics. The use of (Arc View 3.2 GIS) software was actually validated in 2003 [45].

The conclusions were; the applicability of the method over other catchments may have to be looked at in future as well as the need to carry out a hydrological model-ling to investigate the impacts of small reservoirs in water resources in other basin. In general, and with the capacities of these small reservoirs known, planners and wa-ter managers will quickly make decisions on how to utilize and manage the available water given the various competing uses. Moreover, a water manager will in case of flooding be able to predict the likelihood of floods and hence putting in place reme-dial actions to ensure the community is not at risk [46]

Estimation of small reservoir storage capacities with remote sensing in the Brazilian Savannah Region was also conducted and results were published in 2011. This work was carried out for the purpose of efficient water management and sound planning of 147 small reservoirs which can store 19×106 m3 of water at full capacity located in the Preto River Basin which was hindered by inadequate knowledge of the number, storage capacity and spatial distribution of reservoirs in the basin.

To address this lack of data, a simple method to estimate reservoir storage volumes based on remotely sensed reservoir surface area measured with (LANDSAT) was de-veloped. The method was validated by choosing a subset of reservoirs in this basin for which surface areas, shapes and depths were determined with ground-based sur-vey measurements.The agreement between measured and the remotely sensed reservoir volumes was satisfactory indicating that remotely-sensed images can be used for improved man-agement of water in the Brazilian Savannah. The inventory and classification of res-ervoirs was conducted by means of remote sensing using three (Landsat ETM) imag-es taken in 2005. Investigation of the selected reservoirs in the chosen subset con-sisted of visiting the site, collecting information from farmers with questions focused on reservoir characteristics such as existence of technical information (area, depth, maps, etc.), maintenance, age, purpose, etc. The initial database consisted of 252 small reservoirs, but only those with surface areas between 1 and 50 ha were consid-ered. Estimation of water depths was done using a plummet from a boat and (GPS) coordinates determined at many locations inside these reservoir. In order to assure good quality data and whether more readings should be takes, the depth and (GPS) data were downloaded to a laptop and analyzed before leaving the site. Satellite measurements of 75 reservoir surface areas were well correlated with field measured areas. Deviations in surface area measurements between the two methods were the result of both the coarse image spatial resolution of (30 m) and the presence of herbaceous water plant cover in the shallow tail parts of the reservoirs. Surfer Golden Software was used to calculate the surveyed reservoir volumes by creating a 3D-Model for each reservoir, and the volume-area relationships were determined by a linear regression of the log of both the maximum surface area and storage capaci-ty[47]. The general relationship between measured reservoir volumes and their re-motely sensed surface areas showed good agreement. The R2 =83% gives, in some extent, confidence in its use, especially for reservoirs with no other available infor-mation. Combining this relationship with periodic satellite-based reservoir area measurements will allow hydrologists and planners to have a clear picture of water resource system in the Preto River Basin, especially in ungauged subbasins. At full capacity, the water that the Preto River Basin’s 147 small reservoirs can capture is likely small, but these act as a set of well-distributed and easily accessible water source systems that have multiple uses, reducing the population’s vulnerability and improving their livelihoods. The applicability of the derived relationships to other catchments should be examined in future [48].

In 1994 a comprehensive study on the applications of remote sensing methods summed up the application of remote sensing methods required in the field of hydrology, of which many have their use in the investigations of potential dam sites, creating inventories of small dams on regional scale, obtaining volume- area relationships of dams, floods studies and so on. The study indicated the need for more research before operational applications can be made in other areas. The areas requiring additional emphasis that were indicated in this 1994 study were: Land cover, Sediment loads and erosion, Snow water equivalent (complex terrain), Soil moisture, Groundwater, Physiography, Albedo, Evapotranspiration, Areal inputs to models, Streamflow, Infiltration, Chemical pollution.The study concluded that rapid advances in this field were expected in the following several years, especially if adequate effort is devoted by technique developers to technology transfer of the remote sensing methods [49]. This prediction proved its foresight as may be seen from another publication in 2018 which includes details of advances made in the following years in the technology of remote sensing and the versatility of the topics researched [50].

6. Remote Sensing application in Bathymetry studies

Accurate determination of water depth is important both for the purposes of monitoring underwater topography and movement of deposited sediments, and for producing nautical charts in support of navigation. Moreover, bathymetry has proven its value in studies on hydrological and biochemical processes in water bodies such as lakes and reservoirs, in addition to presenting an aid to constructing weather forecast models where temporal variation of surface area and reservoirs depths are taken into consideration. Bathymetry has been traditionally done by vessel based echo sounding and also by airborne remote sensing by (Lidar) by illuminating the target with laser light and measuring the reflection with a sensor which can produce accurate depth measurements. These methods, however, can in situations be costly, inefficient, and inapplicable to very near shore shallow waters and limited by the survey width. By comparison, satellite remote sensing methods offer more flexible, efficient and cost-effective means of mapping bathymetry over broad areas.

This remote sensing bathymetry generally falls into two broad categories [51]:
i)non-imaging
ii) imaging methods

The first category; is based on the use of (Lidar) sensors, that measure distances by illuminating the target with laser light and measuring the reflection. Differences in laser return times and wavelengths can then be used to make digital 3D representations of the target and the method has terrestrial, airborne, and mobile applications [52]. It is able to produce accurate bathymetric information over clear waters at a depth up to 70 m. However, this method is limited by the coarse bathymetric sampling interval and high cost.

The second category; this method can be implemented either analytically or empirically, or by a combination of both. Analytical or semi- analytical implementation is based on the manner of light transmission in water. It requires inputs of a number of parameters related to the properties of the atmosphere, water column, and bottom material. Thus, it is rather complex and difficult to use. Both implementations of this category can produce fine-detailed bathymetric maps over extensive turbid coastal and inland lake waters quickly, even though concurrent depth samples are essential. The detectable depth is usually limited to 20 m. The accuracy of the retrieved bathymetry varies with water depth, with the accuracy substantially lower at a depth beyond 12m and it is influenced by such factors as water turbidity and bottom materials, as well as image properties. With the introduction of new sensors, new data collection technology and the explosion of digital datasets, there has come the ability to develop accurate geospatial products. DEMs and bathymetric navigation surfaces are generated from combining high-resolution and spatially dense, raw digital data with proper vertical reference datum surfaces and are capable of supporting a variety of applications pertinent to the coastal manager. By using (VDatum) software [53] bathymetric and topographic data can be transformed into common vertical datum such as shoreline, or Mean High Water; a seamless DEM can then be created. Data integration allows bathymetric-topographic or bathy-topo (DEMs) to be produced upon which various scenarios can be overlaid, such as sea-level rise, storm surge, tsunami and other inundation scenarios [54]. It is worth mentioning that the airborne (Lidar) has been used to map the bathymetry of thousands of small shallow lakes over the Alaska Slope [55].

Advancements made in recent years and the growth of spaceborne altimetry have shown, however, promise for deriving reservoir bathymetry. An example published in 2018 explained such use which was applied to derive 3 D bathymetry of the lower portion of Lake Mead by combining the time series radar altimetry and (Landsat) data sources, which included the hydrological data and maps based on (Shuttle Elevation) data at multiple scales, and on (Envisat) and Altika radar altimeters, and (Landsat)-based water extents. Lake Mead, created by Hoover Dam on Colorado River, which is located in Nevada, USA, was used as the study case due to the large ground-based data availability, including bathymetric measurements used for evaluations. It is also the largest reservoir in the U.S, with a maximum water capacity of 32.2km3. The focus of the study was the lower portion of the lake, i.e., the Boulder Basin. Figure (9) shows a location map of the study area which has a maximum water capacity of 10.5km3. The objectives of the study were to evaluate different techniques that rebuild reservoir bathymetry by combining multi-satellite imagery of surface water elevation and extent.

DEMs were processed in two distinct ways in order to determine 3D reservoir bathymetry. They were defined as (a) linear extrapolation and (b) linear interpolation. The first one linearly extrapolates the land slope, defining the bottom as the intersection of all extrapolated lines. The second linearly interpolates the uppermost and lowermost pixels of the reservoir's main river, repeating the process for all other tributaries.

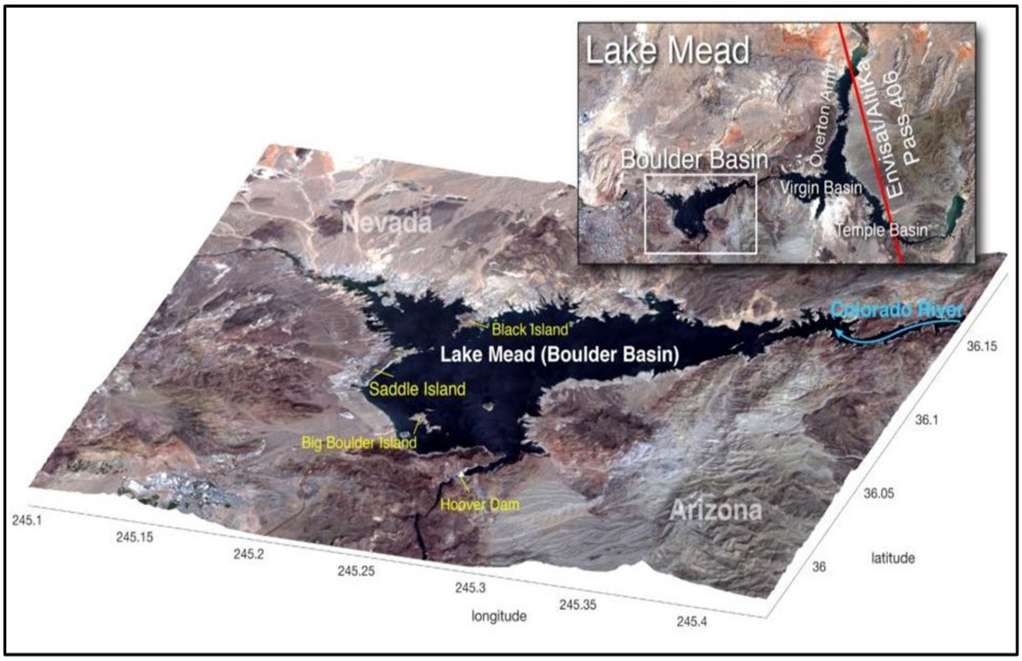


Figure 9. Geographical information of Lake Mead in the USA and Boulder Basin . The Figure is derived from Landsat- 8 OLI image taken on August 14, 2017 [56]

A visible bathymetry, resulting from the combination of radar altimetry and water extent masks, could be coupled with the (DEM) improving the accuracy of techniques (a) and (b). (Envisat) and (Altika) based altimetric time series was combined to a (Landsat) based water extent database over the 2002-2016 period in order to generate the visible bathymetry, and topography was derived from the (3-arcsec HydroSHEDS DEM (i.e. 900m)). Fourteen 3D bathymetries derived from the combination of these techniques and datasets, plus the inclusion of upstream and downstream riverbed elevations, were evaluated over Lake Mead. Accuracy was measured using ground observations, which showed that metrics improve as a function of added data requirement and processing Best bathymetry estimates were obtained when the visible bathymetry linear extrapolation technique and riverbed elevation were combined. Water storage variability was also evaluated and showed that best results were derived from the aforementioned combination.

In this type of studies two limitations , however, were recognized in the evaluation. They were:

i- limited number of the evaluation techniques to generate 3D bathymetry.

ii- limited number of evaluation sites.

Applying and combining additional extrapolation and interpolation techniques (i.e. polynomial, logarithmic, exponential, among others) to a wider range of reservoirs can result in a more refined evaluation, and metrics could be related to hydrogeological features (e.g. shape, depth, etc.) in order to determine the most appropriate techniques to be used. Unfortunately, access to ground-based bathymetric observations is still limited, restricting a broader evaluation [56]. To overcome the limitation of availability of elevation observations from radar altimetry another study was published in 2019 which was made with the objective to explore the potential of generating high resolution reservoir bathymetry at a global scale using data collected by the Multiple Beam Experimental Lidar (MABLE) on board NASA, Ice, Cloud, and land Elevation ICESat-2, which is the 2nd-generation of this series [57]. The study covered again the Bolder Basin forming the western part of Lake Mead shown in Figure (10), (also indicated in Figure (9) above).

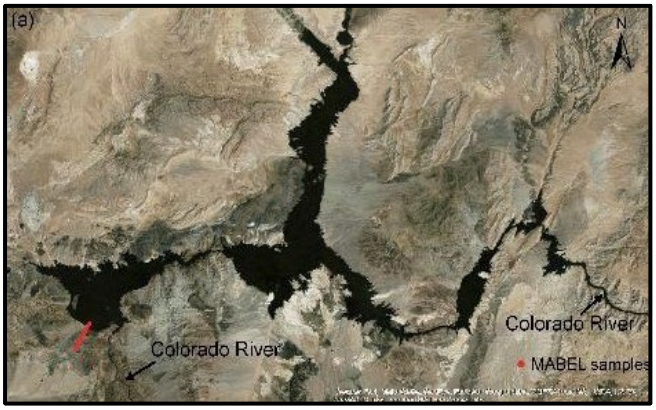


Figure 10: Location map [58]

(MABEL) elevation data were collected on February 24, 2012 , with reported clear sky and relatively low turbidity where the collected data was over approximately 2m footprint along its track. The data was then merged with the Shuttle Radar Topographic Mission (SRTM DEM) to get the final telemetry. Leak Mead (Lidar) survey data collected by the United State Bureau of Reclamation (USBR) in 2009 were used to validate part of the bathymetry product which was generated using (MABEL) data. The methodology can be illustrated by the flow chart in Figure (11).

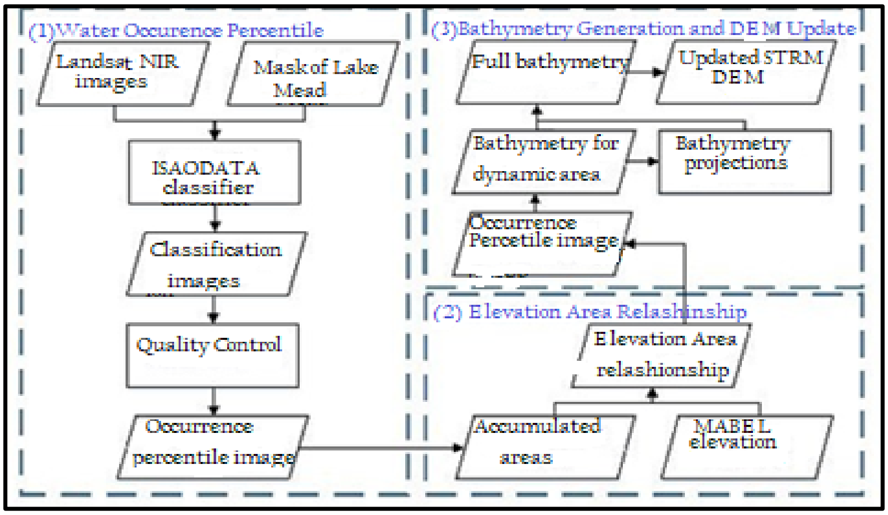

(1)Water Occurence Percentile
Landsat NIR images
Mask of Lake Mead
ISAODATA classifier
Classification images
Quality Control
Occurrence percentile image
(3)Bathymetry Generation and DEM Update
Full bathymetry
Updated STRM DEM
Bathymetry for dynamic area
Bathymetry projections
Occurrence Percetile image
(2) Elevation Area Relashinship
Elevation Area relashionship
Accumulated areas
MABEL elevation

Figure 11. Flowchart of the bathymetry generation algorithm. It consists of three parts (contained in the three blue boxes) : 1) the water occurrence percentile image was generated from the Landsat classifications, which essentially provided the bathymetry contours; 2) the Elevation- Area relationship was established by combining the area- from occurrence percentile image with MABEL elevation values. This was then used to identify and assign elevation values to the contours, and ; 3) The Elevation- Area relationship was applied to the bathymetry for the dynamic lake area. Which was used to project the bathymetry of the dynamic lake for the central area to obtain the full lake bathymetry. Then the full lake bathymetry was overlapped with the SRTM DEM data to replace the constant value [58]

Figure (12a) shows the full bathymetry of Lake Mead, including the remotely sensed part (330.62–369.65 m) and the projected part (263.00–330.62 m). Although the performance of the projected bathymetry varies with region, it has an overall good pattern. Close-up views of the two regions were selected to show the details. In the subset area shown in Figure (12b), the contours based on the projected bathymetry agree well with those from the sedimentation survey data. However, in the subdomain depicted by Figure 12(c), the projected results largely missed those from the survey; which was caused by the assumption made that the slope remains constant. The resultant full bathymetry was then embedded onto the (SRTM DEM) data. The comparison between the bathymetry and (SRTM DEM) is shown in Figure (13). It was evident that the elevation of Lake Mead from (DEM) is constant (372 m) over the entire reservoir. In order to be accurate, the researchers first applied the reservoir area from the (DEM) (587.50 km2) to the (MABEL)-based elevation-area relationship to estimate its corresponding elevation (367.82 m). A systematic bias of 4.18 m was found between this elevation value and that according to the (SRTM DEM). A bias correction of 4.18 m was then applied to the bathymetry such that it could be seamlessly combined with the (DEM). Thus, the part of the (DEM) with a constant value of 372 m was replaced by the part of the bathymetry results representing 372 m and below after the systematic bias correction. Moreover, contour maps with different horizontal intervals (e.g., 5, 10, and 20 m) were drawn from the bathymetry, which can help to delineate the profile. An example of the contour map at a 20-m interval is shown in Figure (13).

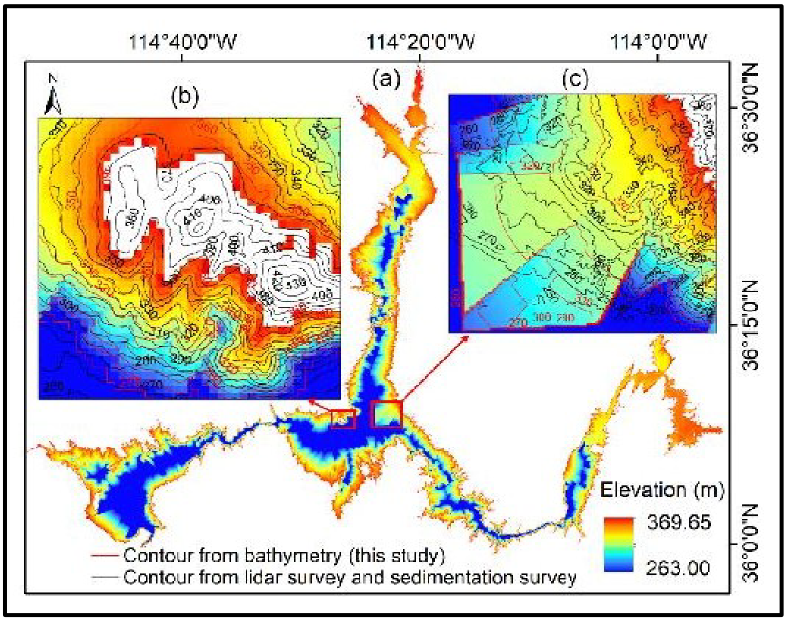


Figure 12: (a) Full bathymetry of Lake Mead, including the remotely sensed bathymetry (3330.62- 369.65m) and the projected bathymetry (263.00- 330.62m). (b), (c) close-up views of the two regions with (b) satisfactory and (c) unsatisfactory performances are selected to show the details. Note that the contour data from both sources have an elevation interval of 10m. The contour data are not shown in (a) because they are relatively dense and would not be useful to the reader [58]

Validation of the remotely sensed bathymetry products showed an overall good agreement when comparing it with available USBR Lidar data and sedimentation data sets. Comparing the statistical bathymetry results obtained from this study with those obtained from the (lidar) survey within each percentile band (at a 10% interval), there was a clear overestimation in the 1%–30% range and an underestimation in the 80%–100% range. However, the derived bathymetry in overall agrees very well with the (lidar) survey data. The overall survey data show a larger variation within each band, which can be attributed to both the higher resolution and the measurement error of the raw data, but it was unclear why the survey elevations in the 90%–100% band are higher than those in the 80%-90% band.

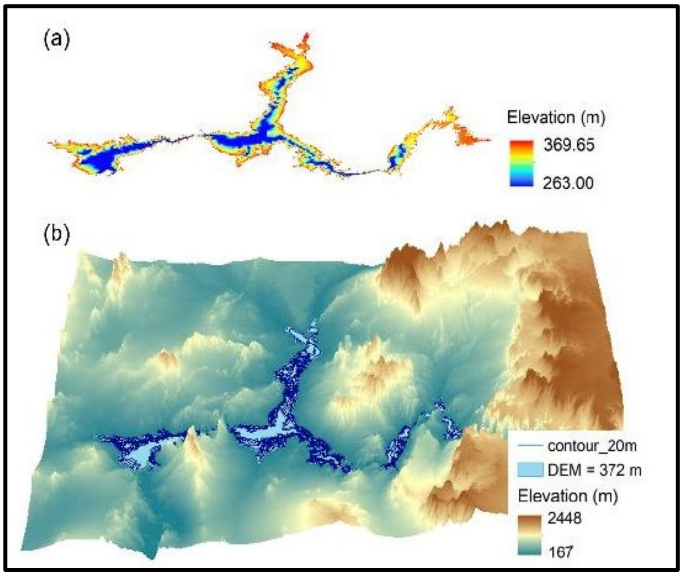


Figure 13: (a) Full bathymetry and (b) STRM DEM data over Lake Mead. The contour map (with a 20m interval) is derived from bathymetry information and DEM elevation is constant (372m) over the entire lake[58].

In summary, it was demonstrated that the 532 nm (MABEL) elevations and imagery data sets can be combined to generate high-resolution 3D reservoir bathymetry results for the dynamic area over the last three decades. The bathymetry for the central reservoir area can be projected by extrapolation and then integrated with the remotely sensed results to obtain the full bathymetry. The highlight is; that a 30m resolution and high-quality bathymetry profile representing the complete range of the dynamic reservoir area over more than 30 years (i.e., 58% of Lake Mead's capacity) was produced exclusively from remote sensing data. The next conclusion was that until the time of this research work, there had been no cost-effective approach to derive reservoir bathymetry on a global scale.

Similar method was used to derive area- elevation relationships for reservoirs on global scale and a 30m resolution bathymetry data set was generated for 347 global reservoirs, representing a total volume of 3123 km3 (50% of the global reservoir capacity). This was documented in a recent study published in 2020 study [59].

First, Area- Elevation (A-E) relationships for the identified reservoirs were derived by combining altimetry data from multiple satellites with Landsat imagery data. Next, the resulting (A- E) relationships were applied to the Surface Water Occurrence (SWO) data from the Joint Research Center (JRC) Global Surface Water (GSW) dataset [60] to obtain bathymetry values for the dynamic areas of the reservoirs. Lastly, an extrapolation method was adopted to help achieve the full bathymetry dataset. The remotely sensed bathymetry results were primarily validated as follows:

i- Against Area-Elevation (A-E) and Elevation-Volume (E-V) relationships derived from the in situ elevations and volumes for 16 reservoirs, with root-mean-square error (RMSE) values of elevation from 0.06 m to 1.99 m, and normalized RMSE values of storage from 0.56% to 4.40% ; and,

ii- They were also validated against survey bathymetric maps for four reservoirs, with R2 values from 0.82 to 0.99 and RMSE values from 0.13 m to 2.31 m.

The projected portions have relatively large errors and uncertainties (compared to the remotely sensed portions) because the extrapolated elevations cannot fully capture the underwater topography. Overall, this approach performs better for reservoirs with a large dynamic area fractions. It can also be applied to small reservoirs (e.g., reservoirs with surface areas of a few square kilometers or less), where (ICESat) observations are available, and to large natural lakes. With the contribution of (ICESat-2), this dataset has the potential to be expanded to thousands of reservoirs and lakes in the future [59].

It is noted that large volume of research work was performed during the last decade on various bathymetry applications utilizing satellite techniques and innovations in monitoring water bodies; ranging from oceans to lakes and reservoirs of various sizes. To cite very few examples, one may mention some of them such as ; (Complete Remote Sensing Approach to Monitor Reservoirs Volume (2011)) [61]; (Bathymetric Mapping Using Landsat8 Scattered Imagery (2015)) [62], (Lake Nubia Bathymetry Detection by Satellite Remote Sensing 2018) [63]. (A New Digital Lake Bathymetry Model Using the Step-Wise Water Recession Method to Generate 3D Lake Bathymetric Maps Based on DEMs, (2019)) [64].

7. Reservoir Siltation and Catchments erosion Observed by Satellite Remote Sensing

Evaluation of sediments volume and its spatial distribution in any reservoir is an important factor in water management of river basins especially those on which dams and reservoirs are constructed. Sediments accumulation does not only affect the useful life of the reservoir and causes reduction of its storage, and hence water availability, whether for irrigation or/and power generation, but it will also affect its operation schedule, in addition to its impact in many cases on water quality. Increased sediment concentration can create turbid waters with a smaller euphotic zone which may have negative impacts on fish and bird species and causing abrasion of fish gills, thus increasing potential for diseases or mortality. Turbidity can also cause visual impairment for predatory fish, affecting their feeding habits. Conducting studies on sedimentation accumulation in any reservoir and its spatial distribution can be performed by conventional and remote sensing techniques. The conventional techniques of quantification of sediment deposition in a reservoir such as hydrographic surveys and the inflow-outflow methods, are cumbersome, costly and time consuming, while on the other hand the alternative possible technique, which is the digital image processing of remotely sensed data is; easy, less costly and subjective of minimum human errors, and can be used for routine assessment of sedimentation in reservoirs. Remote sensing techniques use the fact, that the water spread area of reservoir at various elevations keeps on decreasing with the passage of time and recurrence of floods which bring fresh quantities of such sediments. Remote sensing satellite provides information regarding the water spread area of the reservoir at a particular elevation on the date of pass of the satellite which helps to estimate sedimentation in a reservoir over a period of time. The broad spectral range of available satellite remote sensing devices can provide synoptic, repetitive, and timely information regarding the sedimentation in a reservoir. Reduction, if any, in the water spread area for a particular elevation indicates deposition of sediments at that elevation [64].

Moreover, considering the fact that sediments carried to, and deposited in any reservoir originate from the drainage basin in which the reservoir is located, it follows that understanding of the factors leading to this; such as slope, soil type, land use within the drainage basin is important. The study of the factors contributing to the erosion process can help in taking preventive measures on such drainage basin. These actions may range between improving land use methods, stopping deforestation and enhancing vegetative cover, cultivating on terraces, and constructing silt check structures and control of gulling.

Studies based on remote sensing of the whole watershed area have been carried out in some cases to establishing the factors contributing to erosion and rating them in order to have the full view and the dimensions of the problem. An example of such a study is cited from the case of the Ramganga reservoir (India) and the contribution of its drainage basin to the problem [65]. The Ramganga dam is 128m earth fill dam that was constructed on the Ramganga River, a tributary of the Ganga River during (1961- 1974) and it has a live storage capacity of 2,190 million cubic metres. It generates hydroelectric power and facilitates irrigation. The catchment area of Ramganga basin is approximately 32,493 km2 [66]. The study was performed using historical records of water level elevations from 1988- 2001 and satellite data from (LISS- III) sensor [67] of the Indian Remote Sensing (IRS-1C) satellite launched on 28 December 1995 [68]. The study area was covered in Path 97, Row 50 of the satellite orbit, and the remote sensing data for the following dates were considered; 28 September 2000, 15 November 2009, 9 December 2000, 19 February 2001, 8 April 2001, and 2 May 2001. For the preparation of drainage area and contour map, Survey of Indian topographical maps 53-K/9, 10,11, 13, 14, and 15, 53O/1 and 2 at scale of 1: 50,000 were used.

The Methodology that was followed in the study was carried out into two steps:

i- Watershed prioritization
ii- Study of the reservoir siltation itself.

Watershed prioritization is the process of ranking different micro watersheds of the main watershed according to the order in which they have to be taken up for development.

Holistic integrated planning involving remote sensing, and it was found that (GIS) to be effective in planning on watershed approach [69] [70]. In the above mentioned study, the drainage network and contours of the study area were converted into digital formats based on topographical maps (1:50 000 scale) obtained from the Survey of India as mentioned, and for drainage networking and development of a digital elevation model (DEM) the (GIS) software (ILWIS) - "abbreviated from; Integrated Land and Water Information System"- was used. (ILWIS) has the capabilities to integrate image processing, tabular databases, and conventional (GIS) characteristics [71].

The reservoir catchment was divided into nine sub-basins based on the drainage network. The two parameters considered were: (i) the greenness index, to study the effect of vegetation; and (ii) the brightness index, to study the effect of soil and slope. The function "interpolation from iso-lines in (ILIWS)" was applied to each rasterized contour map to generate the (DEM). Using the (DEM), the slope for each sub-basin was estimated. For computation of other parameters, such as the greenness and soil brightness indices, tasseled cap transformation [72] in the (ERDAS IMAGINE) software was then applied [73].

For the quantification of the volume of sediments deposited in the reservoir, the basic information extracted from the satellite data are the reservoir surface areas at different water surface elevations. With the deposition of sediment in submerged areas of the reservoir, the area enclosed by an individual contour, at any elevation, decreases. Greater deposition of sediment causes greater decreases in the contour area. Using the synoptic satellite data and image interpretation techniques, the surface area of the reservoir at the instant of satellite overpass is determined. The decremental change in reservoir capacity between two consecutive levels is computed using the prismoidal formula. The overall reduction in capacity between the lowest and the highest observed water levels can be obtained by adding the decremental capacity at various levels. It is important to note that the amount of sediment deposited below the lowest observed level cannot be determined using remote sensing data. It is only possible to calculate the sedimentation rate within the particular zone of the reservoir. Hence, the volume of the reservoir below the lowest observed level is assumed to be the same before and after sedimentation. The study concluded that the average sedimentation rate in the Ramganga dam reservoir assessed using remote sensing for the period 1988–2000 was (4.28 × 106 m3 per year). This was in reasonable agreement with the average sedimentation rate calculated from recurrent bathymetric surveys of (4.80 × 106 m3 per year). As such, the study confirmed that using remote sensing techniques represents an economical as well as a practical alternative to the much more expensive and tedious conventional bathymetric surveys. As result of siltation, the reservoir appears to be losing, on average of 0.15% of its initial capacity every year; which is within the normal range of loss in similar environments. The watershed prioritization process carried out as a first step has helped to identify where land management actions were needed most, and the priorities of the most vulnerable sub-watersheds.

In following studies of reservoir sedimentation, many more assessments were performed of various dams in many other watersheds in India. The results of these studies were summarized and published in 2011 [74].The results of 23 reservoirs studies were presented including new study on Ramganga dam reservoir. The storage capacity of these reservoirs ranged between 195x 106 to 10, 287 x 106 m3.The sedimentation rates and capacity loss rates were listed in this new study including the new evaluation of Ramganga dam reservoir sedimentation. The study used water levels data and results of sedimentation surveys from 1974- 2001 historic in addition to the data from (IRS ID, LISS-III sensors) The results of Ramganga dam reservoir revealed a sedimentation rate of (4.23x 106 m3 per year) and capacity loss rate of 0.163% per year; both figures show very close results to those obtained in 2005 which confirm the adequacy of the method and technique used [74]

On the spatially distributed assessment of various sedimentation problems on watershed scale, remote sensing was also used as a powerful tool to assist with other available techniques. In this matter the study on the upper River Citarum Basin in West Java, Indonesia may be mentioned. This study concentrated on the sediment yield and shallow landslide potential area in the upper Citarum River basin which contributed to the silting up of the Saguling dam reservoir located at the outlet of this watershed. The total area of the upper basin is the 2310 km2 lying between altitudes of 600 and 3000 m, and comprising 16 main sub-basins. Geographically, the area is located between 107°26′ E-107°95′ E and 6°73′ S-7°25′ S. The average gradient derived using a 90 m resolution (DEM) ranged from 0.01o in the central area to 31.15° in the northern and southern parts, Figure (14) [75].

This basin was recognized as an area with some of the most persistently active landslides in Indonesia. The floods that trigger the landslides occur almost every year and cause extensive damage. The hydrologic response of the basin has been characterized by land degradation resulting from ; floods, debris flows and others landslide types which were very frequent during the rainy season. The soils derived from volcanic tuff are highly erodible and prone to landslides. Hill slope erosion is also a serious problem in this upper area where hillsides are steep. Shallow landsliding, as a form of mass wasting, and surface soil erosion, are the main source of the basin sediment yield. Therefore, there was an urgent need to devise control measures. Within sediment risk and disaster reduction programs, effective control of sediment mobilization requires implementation of Best Management Practices (BMPs) in the critical sediment source areas of the basin.

Numerous studies have indicated that, for many basins, a few critical locations are responsible for a high proportion of the downstream sediment load. Therefore, it is better to implement control and management measures in the most critical internal locations, as identified at the sub-basin scale, which makes it essential to prioritize individual subbasins. Such prioritization involves ranking of the different critical sub-basins of a main basin according to the order in which they should be targeted for the implementation of structural or non-structural control measures. This procedure was already mentioned in relation to the case of Ramganga study case.

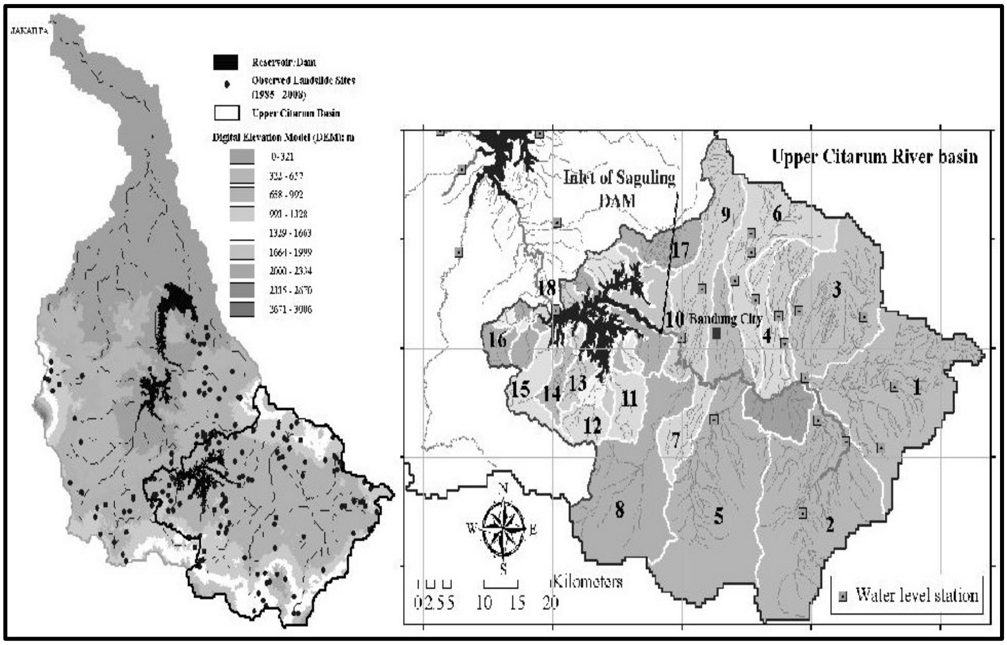


Figure 14: (a) Location of the upper Citarum River basin and the DEM for the whole basin. The filled circles indicate the locations of landslides as recorded by the Geological Agency of Indonesia. (b) Sub-basins boundaries (shown by numbers) and location of water level stations.

The nature of vegetation on the land surface influences both the hydrologic conditions and slope stability. In this study land use information was derived from (LANDSAT7/ETM+ - 30m resolution) satellite images verified by field investigation [76]. A 12-type land-use classification was applied for years 2002 and 2005. Below 800 m, the land use is mainly paddy fields and as settlement areas. At intermediate altitudes and on the steep slopes the land use is mainly farmland (cultivation of annual or tree crops), orchards (perennial trees), forests, grass, and shrubs. Urbanized areas are also found on the steep slopes, although these are relatively small and are concentrated in several small areas which are highly susceptibility to land sliding.

An integrated hydrological- geotechnical model for this assessment was used and it consisted of hill slope hydrology, soil erosion, slope failure, and sediment transport algorithm. Although this model was not named specifically in the study report but we may guess that it could have been the known model called (CRESLIDE)- Coupled Routing and Excess Storage and Slope-Infiltration-Distributed Equilibrium- or similar. Evaluation of the hydrological model performance was carried out through calibration and validation of the hydrological response, using long-term simulations. Adjustment of the final parameters was undertaken by performing Monte-Carlo-type simulations on the basis of best model performance. The years 2004 and 2005 were selected as the calibration and validation periods. Sediment discharge data from the Nanjung Station was used to calibrate and validate the simulated streamflow discharges. The rainfall records at 17 daily rainfall stations, distributed in and around the basin, were utilized to account for the distribution of rainfall. However, only one recording (hourly) rain gauge was available, and daily rainfall data were disaggregated to hourly values using an approach derived from analysis of the various temporal rainfall patterns of heavy rainfall events at the automatic rain gauge located at the BMG Station. The total number of rainstorm events of different duration during the period 1986–2004 was analyzed. It was found that heavy 4-hour rainfalls represented the highest density of rainfall events. The average temporal distribution pattern of heavy 4-hour rainfalls was used as a simple model for generating hourly rainfall series from a daily rainfall. Then the hourly rainfall values were spatially interpolated. The spatial rainfall distribution in the basin was estimated using the Thiessen polygon method. The interpolated and disaggregated data standardized to a (GIS) format served as the input for space- and time-varying hydrological model.

The (DEM) was derived from (HydroSHEDS) products. (HydroSHEDS) is a mapping product that provides hydrographic information for regional and global-scale applications in a consistent format. It offers a suite of geo-referenced data sets (vector and raster) at various scales, including river networks, watershed boundaries, drainage directions, and flow accumulations. (HydroSHEDS) is based on high-resolution elevation data obtained during a Space Shuttle flight for NASA's Shuttle Radar Topography Mission (SRTM) [77]. The conclusion of this study was that such physically based distributed hydrological-geotechnical model could accurately simulate streamflow discharges, soil erosion, sediment yield and the spatial pattern of documented historical landslides. All of this will lead to the overall understanding of the erosion- sedimentation pattern and help in adopting the most suited land management procedures.

In countries which depend to large extent on hydropower generation, it is crucial to adopt management tools to reduce sediment inflow to reservoirs of power generating dams for the elongation of the useful operational life. In Brazil for example this is most important; for, although hydroelectricity may represent nearly 25% of the world's electric power generation, this rate increased to 97% during the 1990s in this country. In Brazil, the option for a hydroelectric matrix was strongly influenced by the presence of large fluvial systems, such as the Amazon River (north), Paraná River (southeast) and São Francisco River (northeast). The Paraná basin has the largest hydroelectric potential in operation in South America and there are more than 150 large reservoirs for power generation in the Paraná River and its main tributaries. It follows that the monitoring of sedimentation in hydropower dams has to be considered as a priority and innovative solutions must be evaluated to improve the management of existing reservoirs.

In this case it was most important to use more efficient and cheaper solution to monitor and evaluate sediment trap efficiencies of these reservoirs than assessing load balance at different point along the river system for computing mid to short-term variations in sediment yield or, repeated bathymetric surveys to calculate the sediment volumes stored at the lakes bottom. Both solutions are extremely time consuming and costly and the protocols used are inhomogeneous.From these considerations and after in-depth studies it was found that optical remote sensing could be efficiently used for such assessments, and a study on the evaluation of using optical remote sensing in this field was published in 2019 [78] which is summarized here.

The optical monitoring of water color or the spectral behavior of water is based on the fact that water color is linked to the presence of optically active components within the water column, and three main components alter the optical properties of water in the visible and infrared wavelength ranges. Significant knowledge had been accumulated on the optical properties of oceanic and continental waters through experimental measurements, modeling approaches and remote sensing methods. Numerous studies have indicated that the red and infrared wavelengths are appropriate for retrieving of the suspended particulate matter (SPM) concentrations. Moreover, It was demonstrated that the optical properties of the main Amazonian rivers were stable enough to support (SPM) monitoring using satellite data; thus, a unique retrieval algorithm may be used regardless of the river and hydrological period considered within a large catchment.

The use of satellite data for water quality monitoring over open water bodies may be hampered by the spatial, temporal and spectral resolution of the images, which can limit the retrieval capacity of the sensor. Therefore, selecting the correct spaceborne platform is critical and depends on the application. In the case of hydro-sedimentary studies, fine temporal resolution monitoring is mandatory because the sediment fluxes vary considerably over time. Thus, spaceborne platforms offering fine revisiting frequency together with fine radiometric calibration and sufficient spectral resolution should be prioritized. Moderate resolution imaging spectroradiometer (MODIS) sensors, have been considered as most appropriate for monitoring (SPM) concentrations in large rivers, such as the Amazon basin. In particular, (MODIS) is capable of monitoring surface (SPM) over more than 10 years at different river gauges (1 km wide or larger) and at intermediate to high levels of (SPM) concentration (10 to 1000 mg per liter).

The objective of the mentioned study was to evaluate the use of (MODIS) remote sensing data for the synoptic monitoring of sedimentation in a series of hydroelectric dams in southern Brazil. It was aimed to demonstrate that satellite data, through the monitoring of spatial and seasonal turbidity variation in reservoirs, provide consistent estimates of sedimentation in reservoirs. To this end, a 13-year time series of satellite data was processed to calculate turbidity at different points over six different reservoirs on the Paranapanema River, a tributary of Parana River in south east of Brazil, shown in Figure (15).

A reflectance retrieval model was calibrated and validated using two independent field turbidity data sets acquired in the catchment. The remote sensing derived turbidity time series retrieved were analyzed as a function of reservoirs' operational characteristics and flow conditions. In particular, the study compared the satellite-derived turbidity decrease with the prediction of a sediment trap efficiency model. A methodology based on remote sensing for indirect assessment of sedimentation processes in reservoirs was then proposed.

In this study, moderate resolution imaging spectroradiometer (MODIS) satellite images were used to quantify the sedimentation processes in the cascade of the six hydropower dams mentioned along a 700-km transect in the Paranapanema River, Figure (15). Turbidity field measurement acquired over 10 years were used to calibrate a turbidity retrieval algorithm based on (MODIS) surface reflectance products.

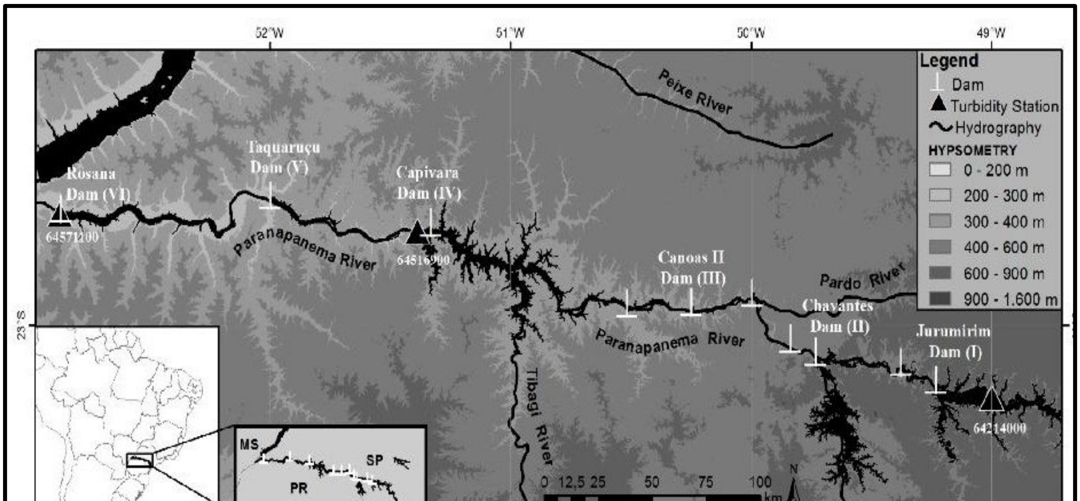


Figure 15: Map of the Paranapanema River catchment showing the location of the hydropower dams (white bars) and turbidity sampling stations (black triangles). The six reservoirs studied are numbered from upstream to downstream (I to VI)[78].

An independent field dataset was used to validate the remote sensing estimates showing fine accuracy (Root Mean Square Error- RMSE- of 9.5 NTU, r = 0.75, N = 138). By processing 13 years of (MODIS) images since 2000, it was shown that satellite data can provide robust turbidity monitoring over the entire transect and can identify extreme sediment discharge events occurring on daily to annual scales. The decrease in the water turbidity as a function of distance within each reservoir, that was related to sedimentation processes was retrieved. The remote sensing-retrieved turbidity decrease within the reservoirs ranged from 2 to 62% making possible to infer the reservoir type and operation (storage versus run-of-river reservoirs).

The reduction in turbidity assessed from space presented a good relationship with conventional sediment trapping efficiency calculations, demonstrating the potential use of this technology for monitoring the intensity of sedimentation processes within reservoirs and at large scale. The study, therefore, fulfilled its objectives and provided more confidence in using remote sensing to assess and monitor sedimentation problems in reservoirs, moreover, it showed the suitability of (MODIS) for such use.

It is interesting to know that about 0.5% to 1% of the total volume of 6,800 km3 of water stored in reservoirs around the world is lost annually as a result of sedimentation. As a result, global per capita reservoir storage has rapidly decreased since its peak at about 1980. Current storage is equivalent to levels that existed nearly 60 years ago. Loss of reservoir storage reduces flexibility in generation and affects the reliability of water supply. Without storage, hydropower facilities are entirely dependent on seasonal flows. These flows might not occur when energy is needed, eliminating one of the key benefits that hydropower provides over other renewables. Sediments discharged from an upstream dam in a cascade system can increase tailwater levels, reducing power generation. This would impact the generation potential of all plants in the cascade and increase the possibility of powerhouse flooding [79].

Many more studies, using remote sensing, have been performed on worldwide scale concerning sediment assessments of individual reservoirs without referring this to erosion in the catchment, such as the Ujjani Dam reservoir study (2017) [80]. This reservoir is located on the Bhima River, a tributary of the Krishna River in Maharashtra state in India. The dam is a combined Earthfill- Masonry Gravity Dam which is the terminal dam on the river and is the largest in the valley. It intercepts a catchment area of 14,858 km2 which includes a free catchment of 9,766 km2.The construction of the dam project including the canal system on both banks was started in 1969 and completed in June 1980. The reservoir created by the dam had a water spread area of 357 km2 at the High Flood Level (HFL) and 336.5 km2 at Full Reservoir Operation Level causing submergence of land and houses in 82 villages. The reservoir stretches upstream of the dam to a length of 134 km, and the maximum width of the reservoir is 8 km. In the mentioned study, (LANDSAT-7) satellite data (resolution 30m) was obtained for the four cloud free dates of water year 2001-

2002 from USGS website i.e. www.usgs.gov.

Digital analysis was performed for identifying the water pixels and for determining the water spread areas. In order for the determination of capacity of the reservoir, reduced levels or water levels were acquired for above four dates from Bhima Irrigation Division, Solapur Maharashtra.

Water Spread areas were found for the four dates from (LANDSAT-7) satellite runs for (27 Oct 2001, 15 Jan 2002, 4 March 2002, 8 June 2002). These were (207.06, 189.69, 178.7, 137.71) km2. The original total design capacity of the reservoir at RL 493.36 was found from the design documentation to be 717.182 million cubic meters, and the reservoir capacity obtained by using Remote Sensing at RL 493.36 in year 2001-2002 was found out to be 509.36 million cubic meters which indicated and annual capacity loss 9.89 million cubic meters during the period of 21 years from 1980 to 2002.

This was judged to be a very high rate of sedimentation which caused concern, so the study called for taking corrective measures in the catchment area to reduce input of silt in the reservoir. One remark on this study is its failure to mention the specification of the sensor on Landsat 7, which was in fact the Enhanced Thematic Mapper Plus (ETM+). Landsat 7 itself was launched on 15 April 1999 and it was designed to last for five years. It had the capacity to collect and transmit up to 532 images per day. It was in a polar, sun-synchronous orbit, meaning it scanned across the entire Earth's surface. With an altitude of 705 km, it took 232 orbits, or 16 days to cove the whole surface of the Earth [81]. More of such studies were carried out in recent years on the use of remote sensing on the assessment of sedimentation of dams reservoirs. Mentioning only two of them as examples we can refer the readers to the study case of Murrumsilli dam reservoir sedimentation (2017) [82], and the Patratu Dam reservoir study (2016)[83].

8. Water Quality Monitoring by Satellite Remote Sensing

Water quality is the general term used to describe the physical, chemical, thermal and biological characteristics of water e.g., temperature, chlorophyll content, turbidity, clarity, total suspended solids (TSS), nutrients, Colored Dissolved Organic Matter (CDOM), dissolved oxygen, pH, Biological Oxygen Demand (BOD), Chemical Oxygen Demand (COD), total organic carbon, and bacteria content. Conventional methods for monitoring water quality parameters by taking in-situ measurement and conducting laboratory analysis are very elaborate, and time consuming. These methods are generally less capable of providing temporal and spatial coverage necessary for the accurate assessment in large water bodies. On the contrary, remote sensing is an effective tool for synoptic monitoring of ecosystem health. Even a few images are useful as aids in the design or improvement of point sampling programs, often through highlighting the best locations and timing for sampling [84].

Application of the remote sensing techniques, due to their capability to provide better spatial and temporal sampling frequencies are gaining importance in the water quality assessment [85]. Figure (16) shows the chlorophyll concentration in the off-coast of California using observation from the (SeaWiFS) and (MODIS) sensors. (SeaWiFS) stand for "Sea-Viewing Wide Field-of-View Sensor" which was a satellite-borne sensor designed to collect global ocean biological data that was active from September 1997 to December 2010, its primary mission was to quantify chlorophyll produced by marine phytoplankton [86].

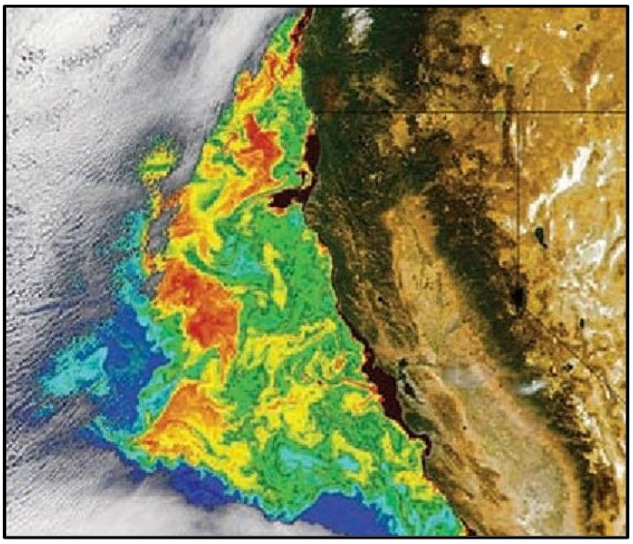


Figure 16: Chlorophyll concentration in the off- coast of California estimated using the SeaWiFS and MODIS sensors. Bright reds indicate high concentrations and blues indicate low concentrations sources [85]

In remote sensing, water quality parameters are estimated by measuring changes in the optical properties of water caused by the presence of the contaminants. Therefore, optical remote sensing has been commonly used for estimating the water quality parameters. Water quality parameters that have been successfully extracted using remote sensing techniques include chlorophyll content, turbidity, secchi depth, total suspended solids, colored dissolved organic matter and tripton. In addition, thermal remote sensing methods have been widely used to estimate the water surface temperature in lakes and estuaries. In remote sensing, optimum wavelength to be used for measuring the water quality parameters depends on the substance that is measured. Based on several in-situ analyses, the visible and near-infrared radiation portions of the EMR spectrum with wavelengths ranging from 0.7 to 0.8 μm were found to be the most useful bands for monitoring suspended sediments in water. Optical properties of water measured using remote sensing techniques are then converted into the water quality indices by using empirical relationships, radiative transfer functions or physical models.

The USBR has been aware of remote sensing applications for very long time, being responsible for managing, developing, and protecting water resources and having particular interest in understanding water characteristics of inland water bodies such as lakes and reservoir. The Bureau has published many reports on the subject including (Remote Sensing Technology: Guidelines and Applications within the Bureau of Reclamation) [87]. This publication explains its use of (Landsat) images for solving water management problems. In the chapter on "Water Studies" it states that the use (Landsat) series could and had been used for studies of some of the properties of water bodies. For illustration purposes, portions of a scene from 2 July 1989, near the Wind River Range in Wyoming, was presented as shown in Figure (17) [87]. At the time, (Landsat) in orbit was (Landsat 5),[88] and it carried same instruments as (Landsat 4) did including the- Thematic Mapper and Multi-Spectral Scanner (TM)-, [89]. The bands were not surface calibrated but served as proxies for the properties. There were several lakes on the southwestern side of the mountain range presented for study that received meltwater from the snow-capped mountains. (TM band 6) gave the surface temperature of the scene; dark was cold and bright was warm. Figure (17) had been colorized over the narrow temperature range of the lakes such that each digital number had a different color, with blue being coldest, red being warmer, then magenta and purple being warmest. The land areas were warmer than the water. In Figure (17) the snow pack was in the upper right and was much darker but with colored edges, indicating that the colored temperatures were close to the melting value. As for the lakes, each lake had a different pattern. The two at higher elevations were cold (light blue). Two have significant temperature patterns within the lake. Notice also that the irrigated agricultural fields in the lower left were cooler (darker) than the barren land.

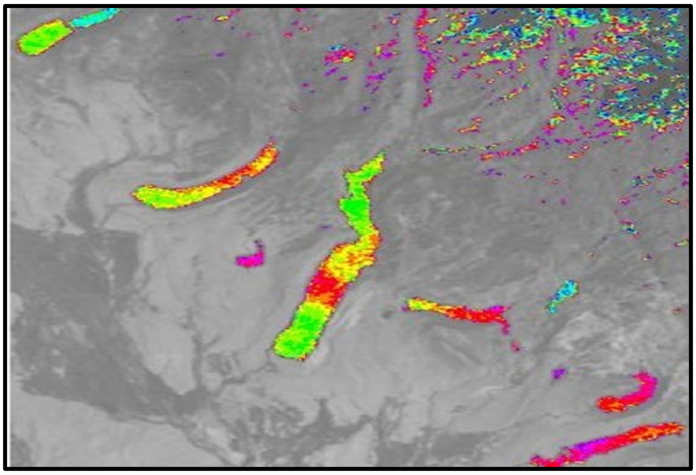


Figure 17: Colorized according to surface temperature [87].

Another area, to the northeast of the Wind River Range, WY, Figure (18) was also selected for further study. The main feature is Ocean Lake, surrounded by an agricultural area with water distribution canals. The natural color version in Figure (18) showed a lightly blue colored lake, suggesting fine sediments from a glacial source in the mountains. This scene was used as a base map for the next views, in which water bodies were colorized to show other water properties.

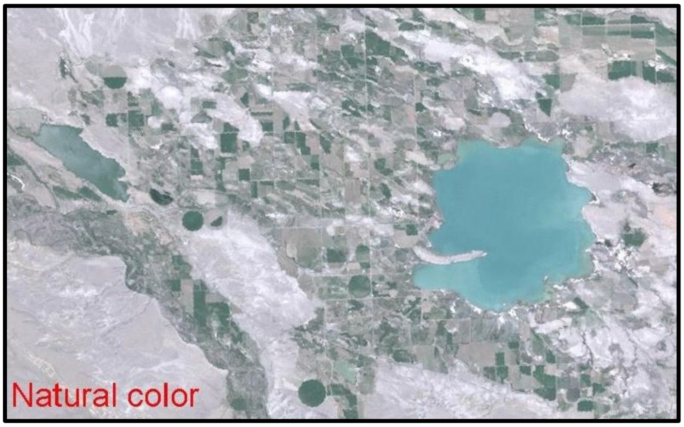


Figure 18. A natural color version of the Ocean Lak, WY [87].

The uncalibrated temperature patterns (TM band 6) showed little variation across the lakes in Figure (19). The western lake was the coldest. The two largest lakes showed warmer (greenish) shorelines, probably because the shallow water let the satellite see to the bottom sediments which are easily warmed by the sun. Along the upper left was a canal with very warm (red) water. The river in the lower left was cooler (green).

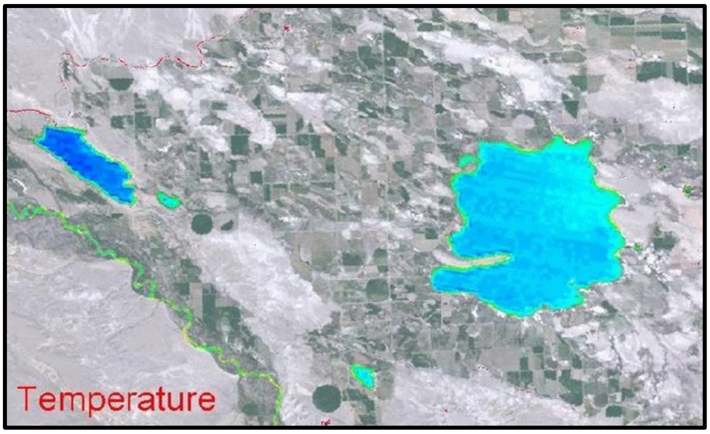


Figure 19: The Ocean Lake scene with water colorized by temperature [87].

Band 1 (blue) was chosen as a proxy for turbidity. The Larger Ocean Lake shows strong patterns as in Figure (20), with red indicating the greatest mud content. There was a small pond at the left that had the cleanest (blue) water. The upper left canal was muddy (reds and oranges) and the lowest river is intermediate (greens). A proper calibration would require field observations of secchi depths; i.e. the depth to which a white disk can be seen in the water.

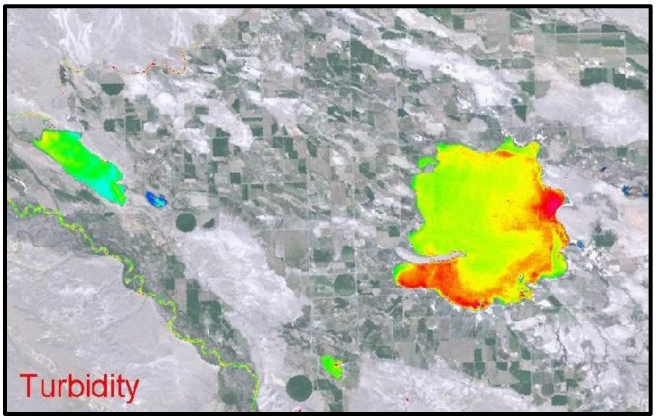


Figure 20: The Ocean Lake scene with water colorized by turbidity [87].

Bands 4 (near-IR) and 3 (red) were combined as a normalized difference vegetative index. This index is greatest for growing vegetation. Though darkened by the water, it serves in Figure (21) as a proxy for chlorophyll content. Reds (small western pond, lower left river) indicate the greater amounts of growth while greens indicate lesser amounts. A proper calibration for chlorophyll makes a regression analysis among the TM bands.

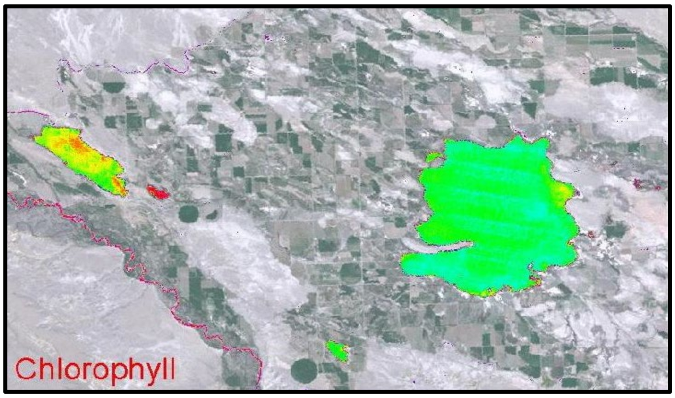


Figure 21: The Ocean Lake scene with water colonized by chlorophyll content [87].

All six reflective TM bands (not band 6) can be combined in a “tasseled cap” analysis to create three mathematically orthogonal bands labeled: Brightness, Greenness, and Wetness. For the Ocean Lake scene in Figure (22) these products are displayed as R, G, and B, respectively. Only a few fields showed a wetness signal. Growing vegetation had variable greenness. Barren areas had the strongest brightness.

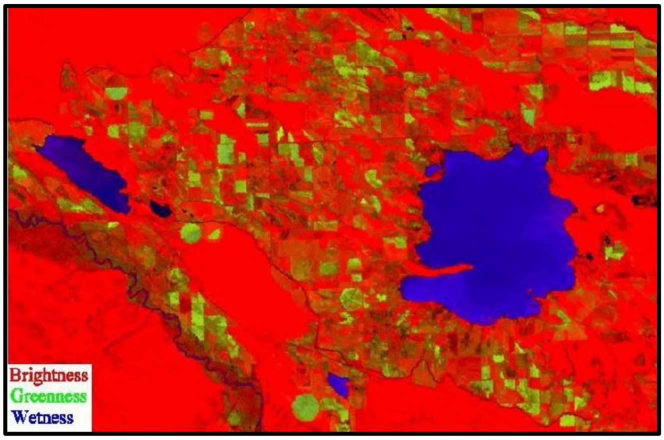


Figure 22: The Ocean Lake scene with false color rendition of brightness, greenness, and wetness [87].

The preceding information serve only as an introduction to the techniques in water quality observations by spaceborne remote sensing as used by USBR in 2005.

But since then there has been great developments in these applications coupled by remarkable progress in space technology.

One review of the research trends in the use of remote sensing that was published in January 2020 explores the progression of inland water quality remote sensing from methodological development to scientific applications; this was based on the examination of 236 key research papers. The results highlight an initial 30 year period where the majority of publications focused on model development and validation, followed by a spike in publications beginning in the early-2000s applying remote sensing models to analyze spatiotemporal trends, drivers, and impacts of changing water quality on ecosystems and human populations. Recent and emerging resources, including improved data availability and enhanced processing platforms, are enabling researchers now to address challenging science questions and model spatiotemporally explicit patterns in water quality.

Examination of the literature shows that the past 10-15 years have brought about a focal shift within the field, where researchers are using improved computing resources, datasets, and operational remote sensing algorithms to better understand complex inland water systems. Future satellite missions promise to continue these improvements by providing observational continuity with spatial/spectral resolutions ideal for inland waters [90].

One of the remote sensing applications, which has been used for long time and has become popular in recent years, is the observation of water quality in water bodies where eutrophication has been one of the most common problems facing the health of lakes and reservoirs. Eutrophication is rapidly increasing due to excessive discharge of nutrient-rich effluents originating from industry and agriculture in basin areas which is primarily causing the production of Chlorophyll-a. This is a specific form of chlorophyll used in oxygenic photosynthesis which absorbs most energy from wavelengths of violet-blue and orange-red light. It also reflects green-yellow light, and as such contributes to the observed green color of most plants [91].

Remote sensing models estimate chlorophyll concentrations by correlating spectral reflectance and reservoir chlorophyll. Different algal populations have different spectral signatures and thus different correlation models; an issue addressed typically by developing and applying models using the same satellite image. The population differences can be exploited by developing seasonal models which can be applied to other images from that season. Algal succession and the assumption that the phytoplankton population is relatively constant over a season are relied upon. In this context, a specific study, authored by Hansen and others, published in 2019 shows how this was applied by dividing the growth season into three parts as substitutes for population measurements. In previous works the same authors had developed seasonal models for Deer Creek and Jordanelle dams reservoirs in Utah (USA) by using (Landsat-5) and (Landsat-7) reflectance measurements, but in the present study the previous work was generalized by demonstrating regional application using models developed with a more extensive dataset. The authors show how this approach can be used to develop a long-term record of water quality in several reservoirs from available historic (Landsat) data. They used data from the two reservoirs to develop seasonal models, then applied these models to similar regional reservoirs without sufficient field data for model development. Because the seasonal models can be applied to other satellite collections that occurred in the same season, this approach supports chlorophyll estimates to define long-term seasonal trends, provides frequent updates on reservoir conditions, supports comparison among reservoirs, and may be used for short-term forecasting [92].

The sampling locations for Deer Creek and Jordanelle reservoirs (green dots) used for the study are shown in Figure (23). (Landsat) image from 14 July 2014 shows the five study reservoirs in Utah; this is presented in Figure (24). Deer Creek Reservoir is located on the Provo River and supplies potable water for much of the Salt Lake City valley. The reservoir has a surface area of 1200 ha. with a capacity of about 240 million m3 and a maximum depth of 42m near the dam. Deer Creek Reservoir was identified as highly eutrophic in the 1970s and 1980s and continues to exhibit high phosphorus and low dissolved oxygen levels during the summer. Jordanelle Reservoir is a large reservoir with surface area of about 1335 ha. and feeds into Deer Creek Reservoir. It has a capacity of approximately 445 million m3 and a maximum depth of 89m. The dam was completed and the reservoir completely filled in 1994.

The other three reservoirs to which the results of the study were extended were:

i-East Canyon Reservoir; a 276 ha reservoir located behind the Wasatch Mountains. It has a capacity of approximately 63 million m3 and a maximum depth of 60m. The surrounding watershed is relatively dry compared to the other watersheds in the area. One of the main concerns for this reservoir is the rapid urbanization and development in the contributing watershed.

ii- Rockport Reservoir; a 481 ha impoundment on the Weber River, with a capacity of 93.4 million m3 and a maximum depth of 45 m. Although the historical water quality of this reservoir has been good, conditions have exceeded standards for phosphorus and dissolved oxygen, issues that are closely related to algal biomass.

iii- Echo Reservoir; also, on the Weber River, located north of Rockport Reservoir. It is comparable to Rockport in size at 564 ha, 91.1 million m3 in capacity, and a maximum depth of 33.5m, (Figure (24).

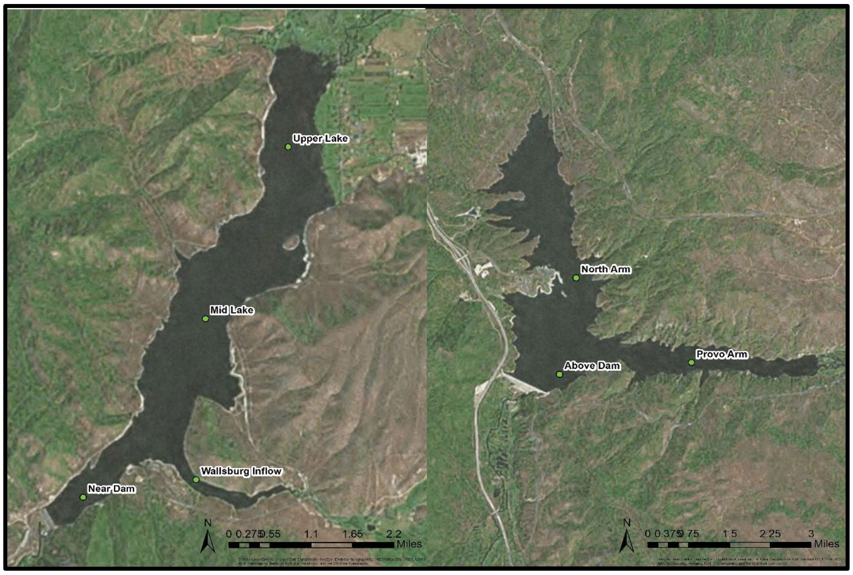


Figure 23: Sampling locations (green dots) for Deer Creek and Jordanelle Reservoirs [92].

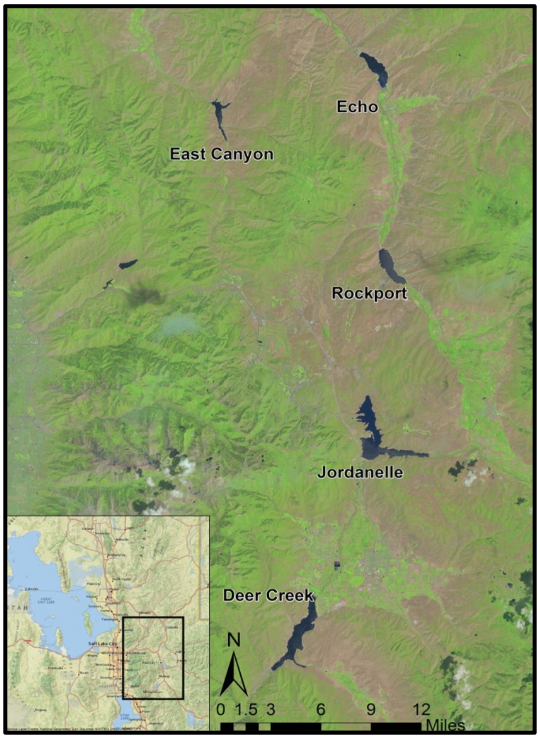


Figure 24: Landsat image from 14 July 2014 showing the 5 study reservoirs in Utah [92].

The methodology used in the analysis was to follow a stepwise procedure by first collecting the archived data for the first two dams for the years from (1984- 2012) for Deer Creek reservoir, and from (1993- 2012) for Jordanelle reservoir. These data had been collected by (Landsat 5- TM mapper) for the year (1984- 1999) and (Landsat 7- ETM +) for the years (1999- 2012). The seasons were split into three periods, each one according to the relative similar phytoplankton communities present in the reservoirs. Phytoplankton is defined as, a flora of freely floating, often minute organisms that drift with water currents. These periods were (April- May), (June- July), (August- September). Following this, the (Landsat) images were radiometrically calibrated and atmospherically corrected by selecting a group of 9 pixels centered on each field sample location and computing a mean filtered average reflectance for bands 1-5 and 7. This procedure was meant to remove noise. Models development for each of the two reservoirs for the three periods was done using multiple least square approach in which a model form is selected from a least square fit to determine model coefficients, and for that a combination of (Landsat) bands 1, 2, 3 and 4 were used. For Models application, the (ENVI, v4.7)- (Exelis Visual Information Solutions, Boulder, CO) was used as the base software platform for applying the models to historical images. The validation of the procedure was done by using the models obtained for each of the three periods for Deer Creek and applying them to corresponding period in another season and comparing the results with field measurements. The results showed good agreements between models of the same periods of different seasons, but when an all-season model was developed for this reservoir, this model showed poor agreement with field measurements for early and mid-season data, and the fit was better for the late season data, although still not as close as the season specific model.

These results confirmed the suitability of the obtained (Periods Models) , but not the (All-Season Model) to application on all other reservoirs with the possibilities of forecasting conditions from just referring them to satellites images that would be taken in future dates.

For the five reservoirs used in this case study, it was observed that each of them demonstrated unique behavior, both historically and within the growing season. Managers can use this knowledge to identify which reservoirs require additional monitoring or mitigation efforts. Because the (Landsat) data provide full spatial coverage, they may better identify which areas of a reservoir should be targeted for field sampling.

Generally, while estimated concentrations using this approach may not be precise, changes in concentration over time and spatial distributions are relatively accurate, implying that even reservoirs without field data can be monitored as long as the optical characteristics of the reservoirs are similar.

In similar study performed in 2019, it was established that monitoring water quality of small water bodies can be done using High- Resolution Remote sensing data. The study was conducted on the Bora-bey dam reservoir in Turkey [93].

Water quality in this relatively small dam was assessed using high-resolution satellite data from (RapidEye) and in situ measurements collected a few days apart. (RapidEye- AG), was a German geospatial information provider focused on assisting in management decision-making through services based on their own Earth observation imagery. The company operated a five-satellite constellation producing 5-meter resolution imagery. Today, (RapidEye) refers to the constellation of 5 earth observation satellites owned and operated by Planet Labs [94].

The satellite carries a five-band multispectral optical imager with a ground sampling distance of 5m at its nadir and a swath width of 80 km. Several different algorithms were evaluated in the study using Pearson Correlation Coefficients for electrical conductivity (EC), total dissolved salts (TDS), water transparency, water turbidity, depth, suspended particular matter (SPM), and chlorophyll-a. The results indicated strong correlation between the investigated parameters and (RapidEye) reflectance, especially in the red and red-edge portion with highest correlation between red-edge band and water turbidity (r2 = 0.92). Two of the investigated indices showed good correlation in almost all of the water quality parameters with correlation higher than 0.80. The findings of this study emphasized the use of both high-resolution remote sensing imagery and red-edge portion of the electromagnetic spectrum for monitoring several water quality parameters in small water area.

It may be concluded that available procedures coupled with available previous and future satellite images can present wide applications on monitoring reservoirs, inlands lakes, and open seas water quality issues. In reservoirs and lakes problems like turbidity, salinity, eutrophication, contamination with tailings and sewerage can be investigated by following variations in Water Quality indices. For open seas, the same can be done in addition to such things as following oil spills impacts, changes in coral reefs and fish habitats and all sorts of changes in the hydro ecosystems. This can be done by using proper satellite images and developing the procedures and models applicable to the intended use. The future will certainly bring new satellites technological innovations. Looking just a little ahead launching (Landsat 9) in mid-2021 will open new horizons in this field. Landsat 9, like (Landsat 8), will have a higher imaging capacity than past (Landsat satellites), allowing more valuable data to be added to the (Landsat)'s global land archive [95].

9. Assessment of Reservoir Evaporation Losses by Remote Sensing

Evaporation from open water surfaces is a major element and continuous process in the global hydrological cycle. With respect to reservoirs, quantities of water lost to evaporation have to be accounted for in the water management of each reservoir as an unredeemed loss. On a world scale these losses are estimated to be more than the combined consumption from industrial and domestic water uses. For example, the evaporation from agricultural irrigation reservoirs in a semiarid basin in Spain is equivalent to 27% of the total domestic water use in this basin [96]. It is estimated that half of water stored in Australia's reservoirs is lost due to evaporation [97].

Accurate estimation of evaporation losses are, therefore, very important to the evaluation of available water resources of a given basin, region or even individual reservoirs. No continental consistent and locally practical evaporation dataset have been produced so far that can be used in the policy making process at a regional scale. To precisely quantify the evaporation losses from a given reservoir, water surface area and evaporation rate data are needed. However, both high quality reservoir surface area and evaporation rate data can be difficult to gather [98]. Regarding evaporation rate, a variety of methods are available, ranging from local-scale approaches such as the use of eddy covariance tower and scintillometer to regional-scale energy balance methods that are based on water surface temperature [99][100][101].

Nevertheless, the use of evaporation pans remains the most popular method for water resource managers despite its sensitivity to local weather and landscape conditions.

Regarding reservoir area, which is a controlling factor, the reservoir area-elevation curve is a widely-accepted approach to estimate the fluctuation of reservoir area. However, reservoir curves could be unavailable if the reservoir is in a data-sparse region, or questionable if long-term sedimentation has changed the original elevation-area relationship. The error in reservoir area estimates is largely neglected in analyzing the uncertainties of reservoir evaporation.

Research in the Satellite Sensing applications to the estimation of evaporation losses is still in its early beginnings. Researchers in recent years have become interested in the use of remote sensing products as a direct mean in the assessment of evaporation losses from lakes and reservoirs, but much is still needed to be done before generalizing their use. One group explored the wide use of the satellite-derived (MODIS) dataset in studies on evapotranspiration, but this study does not cover rivers, lakes, reservoirs, and wetlands [102].The study was designed to describe evapotranspiration from vegetated land surfaces and its input includes Leaf

Area Index, which is not applicable for water bodies.

Another study, however, demonstrated the possible use of (MODIS) as a good tool for evaporation losses assessment [103]. The study obtained the seasonal variation in evaporation for the five major lakes in the Savannah River Basin (in South Carolina, USA). These were; Lakes Jocassee, Keowee, Hartwell, Russell, and Thurmond. The Savannah River Basin (SRB) originates in the Blue Ridge Mountains of Georgia, North Carolina, and South Carolina. The basin has a total drainage area of 27,400 km2, of which 11,900 km2 are in South Carolina, 15,100 km2 are in Georgia, and 453 km2 are in North Carolina. The basin serves as a major water source for more than 1.5 million people within Georgia, South Carolina, and North Carolina as well as possessing a total power generating capacity of approximately 10,661 MW for residential and commercial use. A significant part of the upper Savannah River is controlled by the United States Army Corps of Engineers which operates three multipurpose reservoirs: Lakes Hartwell, Russell, and Thurmond. Duke Energy Company has constructed three additional reservoirs above Lake Hartwell as part of its Keowee-Toxaway project: these are Bad Creek Reservoir (a small pumped storage reservoir located above Lake Jocassee), Lake Jocassee, and Lake Keowee. Full pool elevations of Bad Creek, Jocassee, Keowee, Hartwell, Russell, and Thurmond are 704, 338, 244, 201, 145, and 101 (m.a.s.l), respectively. Geometric data for these five reservoirs are presented in Table (3). A map of the basin and its main reservoirs is presented in Figure (25). The volume of Bad Creek Reservoir and a few other small lakes are negligible, compared to the remaining five lakes in the (SRB), and only those five lakes are considered in the study.

Table 3: Geometric information on the five major lakes in the Savannah River Basin [103].

Lake	Surface area (km^2) (a, c)	Mean depth (m) (b)	Max depth (m) (b)	Shoreline length (km) (c)	Volume (Mm^3) (a, c)
Jocassee	30.6	48.1	99.4	121.0	1489
Keowee	75.0	16.0	90.5	482.8	1072
Hartwell	226.6	14.0	53.6	1548.2	3132
Russell	107.8	12.1	44.8	869.0	1262
Thurmond	283.3	11.3	43.3	1931.2	3072

a) Calculated at full pool elevation
b) Data supplied by South Carolina Department of Health and Environmental Control (SCDHEC)
c) Data supplied by USACE

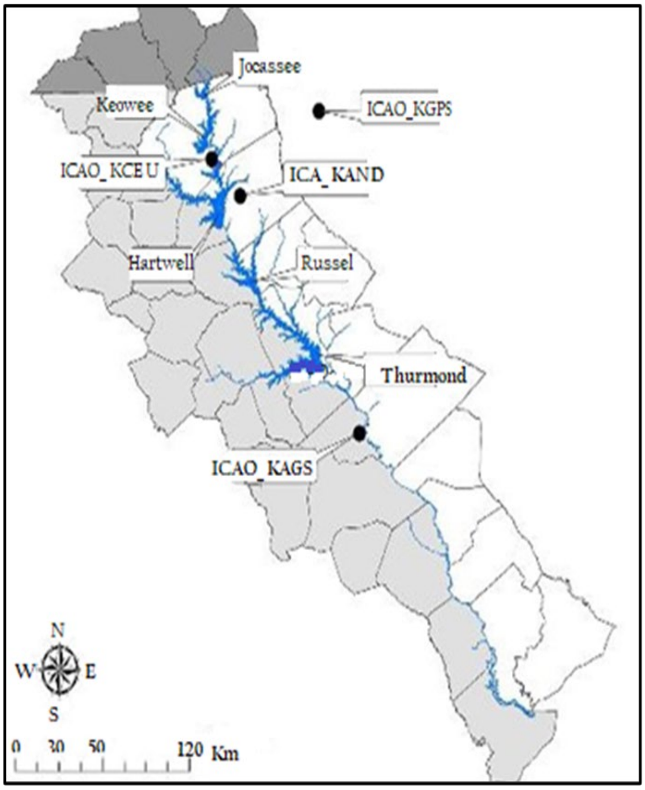


Figure 25: Map of the Savannah River Basin, including major lakes, and nearby regional airport weather Stations (ICAO). North Carolina, South Carolina, and Georgia counties are represented as dark, gray, white and light gray, respectively [103].

The study used data from two sources:

1- Satellite (MODIS) data for obtaining spatially resolved lake surface temperature (Ts) were gathered from (MOD11 A1) and (MYD11 A1). The Land Surface Temperature (LST) products were generated from (MODIS Thermal bands 31 (11 µm) and 32 (12 µm)). The LST (MODIS) data was pre-processed to account for atmospheric and surface emissivity effects. Also, a cloud mask was incorporated for inland water estimates, providing a surface temperature measurement when there is a 66 % or greater confidence of clear-sky conditions.

LST were obtained in the form of daily (MODIS) land surface temperatures, as (LST) level 3, 1-km nominal resolution data for Terra (MOD11A1, version 5) and Aqua (MYD11A1, version 5). Water Surface Temperature (Ts) data was obtained by combining measurements from the (MODIS) sensor on (NASA)'s (Terra) and (Aqua) satellites [104]. These data were available from the (NASA) Earth Observing System Data and Information System (EOSDIS) from July 2002 to December 2012.

The LST product is able to provide water surface temperature. Also, the LST from the (Aqua) and (Terra) platforms can be intercompared without correction. (Landsat 7-ETM) imagery was used to develop a shoreline mask, which was used to exclude any (MODIS LST) pixels-containing land. The number of land-free pixels were 6, 1, 12, 3, and 19 for lakes Jocassee, Keowee, Hartwell, Russell, and Thurmond, respectively. On-site measurements of (Ts) were not available for the lakes studied, and so validation of the (MODIS) data used herein was not possible for the lakes studied. However, validation studies of (MODIS) (Ts) measurements of lake surfaces had been performed by several other authors, which shows the Root Mean Square (RMS) and bias errors between the (MODIS) measurement and in situ measurement for the lake studied were not significant, suggesting the reliability of this approach

2- National weather Service meteorological stations (NWS) supplied the land temperatures measurements (Ta), wind speed (ū), eddy flux (Ø) and relative humidity. These (NWS) stations were belonging to nearby regional airport weather stations (ICAO) shown in the map of Figure (25). These data were used to obtain a mass transfer coefficient (hm) whereby the evaporation rate (E) can be quantified from the respective equation.

The methodology of the study was to compute in two approaches;
i- the evaporation rates obtained from Mass Transfer Methods based on (MODIS) obtained data.
ii- to compare the results with those results of evaporation loss rates calculated from Pan Evaporation measurements.

In the first approach the “Harmonic ANalysis of Time Series (HANTS)” [105] algorithm was used to get the time series of (hm) from the Temperature Time series. These in their turn were obtained by applying three general Mass Transfer Methods namely; the turbulent boundary layer (TBL) Method, the general aerodynamic (AERO) Method, and the heat transfer (HT) Methods. The Temperature Time Series themselves were based on (Ts) values from the spatially averaged (LST) data, and the corresponding hourly(Ta), (Ø), and (ū) measurements from the meteorological station closest to each of the respective lake. The evaporation rates (hm) obtained by the TBL, AERO, and HT methods and (HANTS) were computed by scaling the hourly net evaporative flux by the water density calculated at (Ts) so that all methods report evaporation in units of length per unit time. Finally, the four hourly evaporation.

In the second approach, several sets of Pan Evaporation measurements were available throughout the Savannah River Basin. However, Pan data from the (Class

A) pan located in Clemson, SC was the only available data set with measurements dating back to the development of the Free Water Surface (EWS) grids (1956–1970), as well as covering the period of record for this study. As a result, these daily Pan evaporation data measured in Clemson, South Carolina were used. These were obtained from National Oceanic and Atmospheric Administration (NOAA)'s National Climatic Data Center (NCDC). Based on this, a monthly pan coefficient was developed for each of the lakes within the basin through the use of the Free Water Surface (FWS) evaporation estimates from NOAA's digitized evaporation maps and the Pan measurements obtained from the Clemson (Class A) pan.

As stated before the goal of the study was to investigate the seasonal variation in lake evaporation in the (SRB) using two different methods for measuring evaporation, namely; the three Mass Transfer Methods based on (MODIS) data and the Pan measurements Method. These methods were used to calculate four monthly averaged evaporation totals for each lake. The data clearly showed a significantly different seasonal behavior between the Pan data and the mass transfer methods. For all five lakes, the Pan data showed a peak in early to midsummer, while the mass transfer methods showed a range of seasonal behaviors, none having a peak in the summer. Most of the lakes showed two peaks in evaporation, one in the spring and one in the fall. The exception was Hartwell, which exhibited a single peak in the fall.

The derived Yearly Lake Evaporation corresponding to (MODIS) period of record for each mass transfer approach and for Pan evaporation results are shown for each of the five lakes in Figure (26). Water resources managers in the United States seeking to use the approach presented herein, but wanting to have results that provide similar yearly average totals to historical pan measurements, should use the historical (MODIS) and the Automated climatological observing network in the United States (ASOS) data to establish which mass transfer method best matches their Pan data. For the (SRB), this would be either the (TBL) or (HT) methods. Given the absence of absolute ground truth and the known issues with pan measurements, such a comparison should not be taken to indicate which mass transfer method is the best in an absolute sense.

Future research would enable further development of this method. For example, the application of the method described to a more instrumented lake could be used to investigate the impact of using remote sensing data on the resulting evaporation measurements. Measuring the meteorological conditions on the lake shore or lake surface could be used to quantify the potential error associated with using remote weather station data. Likewise, local measurement of lake surface temperature could be used to assess the quality of the (MODIS) data. These measurements could also be used to validate the use of the (HANTS) algorithm for filling in missing

(MODIS) data. Comparison with on-lake measurements of evaporation using, for example, an eddy co-variance probe or scintillometer would also better parameterize the remotely sensed evaporation measurements against more direct local measurements. In fact, given the availability of historical data going back to June 2002, this approach could be compared with previously published evaporation studies.

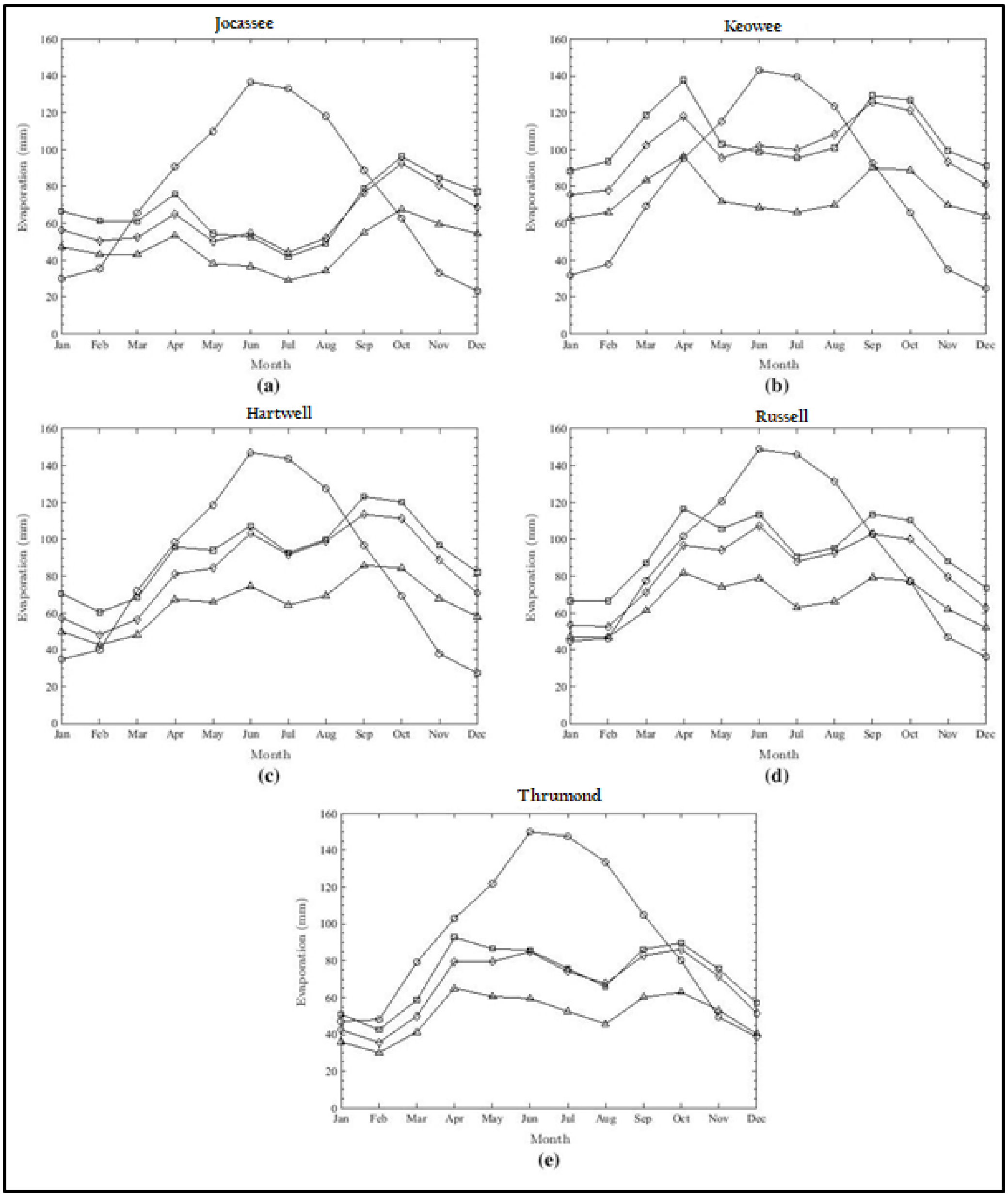


Figure 26: Yearly lake evaporation corresponding to MODIS period of record for TBL (square), AERO (triangle), HT (diamond) and Pan (circle): (a) Jocassee; (b) Keowee; (c) Hartwell; (d) Russell; and (e)Thurmond [103].

The researches claim that since this approach requires only an existing satellite resource with global coverage and existing (NWS) stations, this method can potentially be ported to any lake where there is a nearby meteorology station. Hence, this method could be used by both water resource managers and limnologists alike with the possibility of extending this approach beyond a single basin to encompass an entire geographical region or continent

Another study on remote sensing method for estimating regional reservoir area and evaporative loss was published in 2017 [106]. The researches propose a remote sensing framework to estimate reservoir evaporative loss at the regional scale. This framework uses a multispectral water index to extract reservoir area from (Landsat) imagery and estimate monthly evaporation volume based on pan-derived evaporative rates. The optimal index threshold is determined based on local observations and extended to unobserved locations and periods. Built on the cloud computing capacity of the Google Earth Engine (GEE), this framework can efficiently analyze satellite images at large spatiotemporal scales, where such analysis is infeasible with a single computer. The study involved 200 major reservoirs in Texas, captured in 17,811 Landsat images over a 32-year period Figure (27).

The results show that these reservoirs contribute to an annual evaporative loss of 8.0 billion cubic meters, equivalent to 20% of their total active storage or 53% of total annual water use in Texas. At five coastal basins, reservoir evaporative losses exceed the minimum freshwater inflows required to sustain ecosystem health and fishery productivity of the receiving estuaries. Reservoir evaporative loss can be significant enough to counterbalance the positive effects of impounding water and to offset the contribution of water conservation and reuse practices. Our results also reveal the spatially variable performance of the multispectral water index and indicate the limitation of using scene-level cloud cover to screen satellite images.

The selected reservoirs are all visible on Landsat imagery with cloud cover below 10%. Their total conservation capacity is 40.9 billion m3 and accounts for 79% of Texas's total capacity. Monthly evaporation and precipitation rates of reservoirs are obtained from the Texas Water Development Board (TWDB) in one-degree latitude by one-degree longitude quadrangles, Figure (28).The dataset is based on daily precipitation rates measured at 2,400 precipitation stations and daily evaporation rates at 76 pan evaporation stations across Texas (TWDB, 2016). The daily gross pan evaporation rates are first summed into monthly pan evaporation rates at each evaporation station. Then Thiessen polygons are developed based on the locations of all evaporation stations. The quadrangular pan evaporation rate is calculated as the sum of pan evaporation rates of all stations weighted by the areas of their Thiessen polygons in the quadrangle. It is further converted into a quadrangular reservoir evaporation rate by multiplying by pan-to- lake coefficients, which are developed by the National Weather Service at the monthly level and are specific for each quadrangle. The quadrangular precipitation dataset is produced in a similar way except that there is no need for pan-to-lake precipitation coefficients.

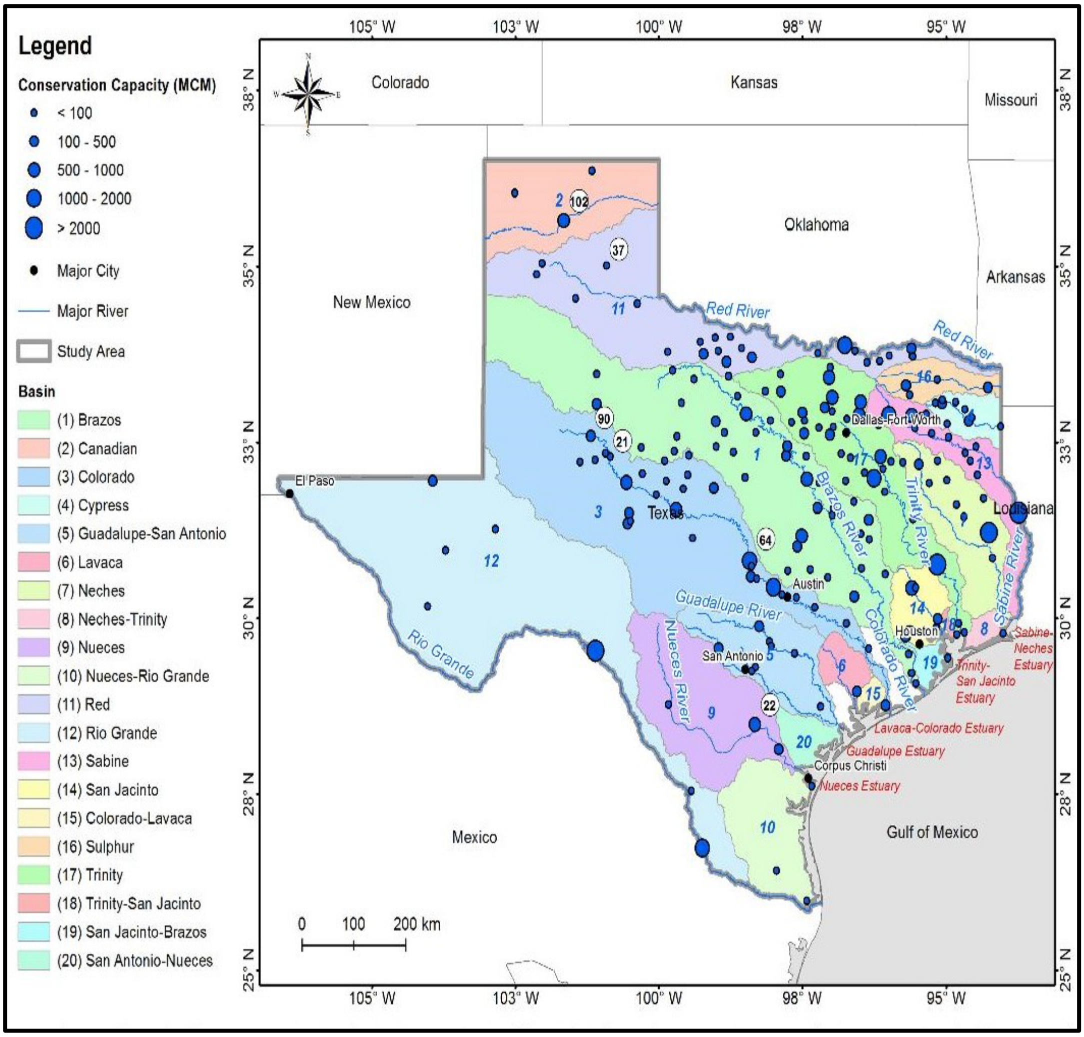


Figure 27. Study area, Blue circles show 200 reservoirs in 20 basins in Texas. The size of the circles indicate the capacity) active storage volume) of the reservoirs [106].

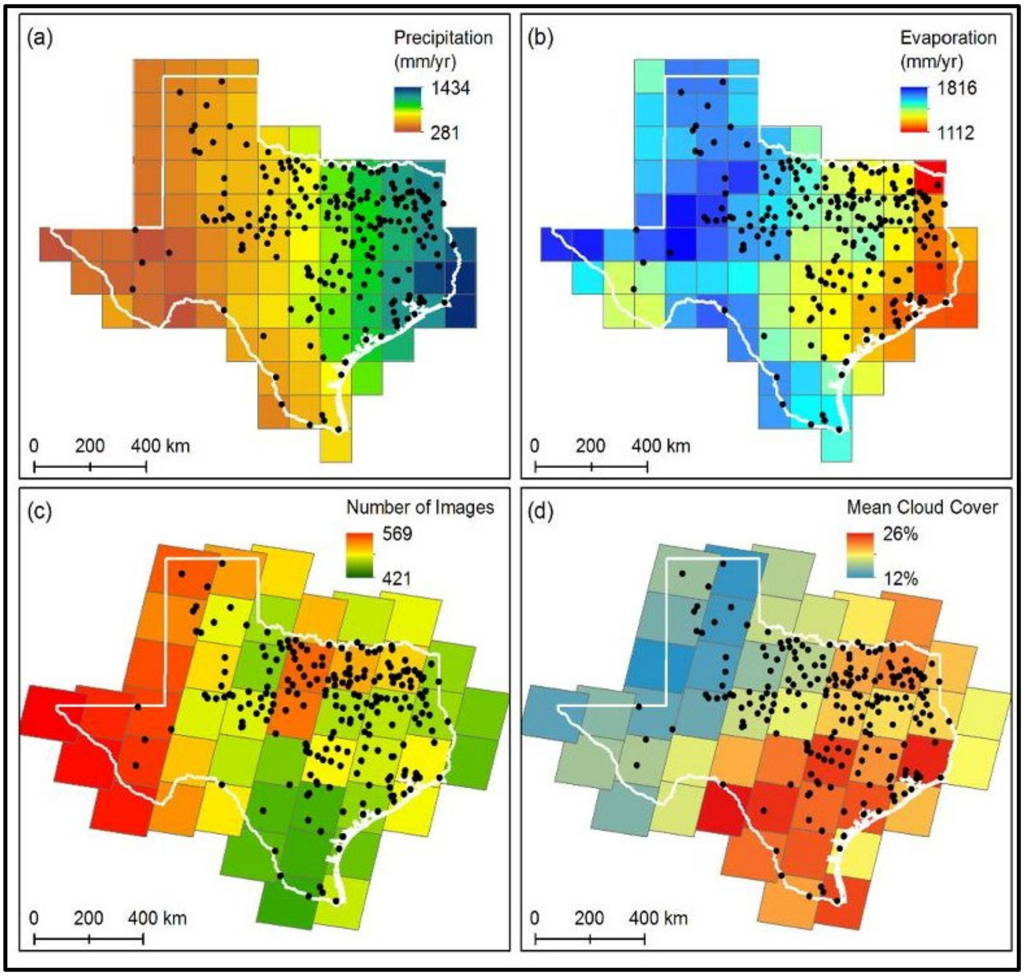


Figure 28: Ground observation data and satellite imagery of the study area: (a) quadrangle precipitation rate, (b) quadrangle evaporation rate in (c) and (d) indicate Landsat WrS-2 scenes [106].

Satellite data used in this study are surface reflectance derived from (Landsat 5) and (Landsat 8) imagery. (Landsat 5 TM) images have six 30-m visible bands and a 120-m thermal band. (Landsat 8 OLI) images have eight 30-m bands, two 100-m thermal bands and a 15-m panchromatic band. The study used a total of 17,811 Landsat images across 47 (WRS-2) scenes of the Worldwide Reference System (WRS) [107] including 15,748 (Landsat 5 TM) images from 1984 to 2011, and 2,063 (Landsat 8 OLI) images from 2013 to 2015. The gap in data in 2012 was filled by using the surface reflectance data produced by the U.S. Geological Survey (USGS) using the Landsat Ecosystem Disturbance Adaptive Processing System (LEDAPS) and accessed through Google Earth Engine (GEE).

Without going into the cumbersome details of the study the methodology follows the flow charts shown in Figure (29), and for details on the modified normalized difference water index (MNDWI) the reader is referred to [108].

The study presented estimates for six reservoirs monthly areas variation (1984-2012) and including Landsat images, (MNDWI), and water pixels. Average monthly evaporation and net evaporation for 20 basins were extracted and presented in graphical and tabular forms. Comparison of satellite derived reservoir evaporative loss to observed reservoir storage change per month for (1984- 2015) at eight reservoirs were also given.

This study demonstrated the advantage of a Google Earth Engine (GEE)-based water resources assessment formwork that can process geospatial big data to assess regional reservoir evaporative loss. This study also confirmed (MDNWI) as an effective spectral index for water identification and revealed the spatial heterogeneity of (MNDWI) thresholds at the regional scale. It is important to note that this method is not developed to estimate evaporation in an area where all reservoirs are totally ungauged. However, this method is capable of generating a satellite-based direct quantification of reservoir area dynamics, completing or extending the current area estimates, expanding the estimates to ungauged reservoirs that are close to gauged reservoirs.

The amount of evaporative loss reflects the combined effects of various engineering, geological and meteorological factors. Understanding the spatiotemporal patterns of reservoir evaporation provides important information for designing effective evaporation suppression measures, evaluating water infrastructure performance and improving regional water resilience.

Results of this study indicate that the magnitude of water loss through evaporation is comparable to water use of major economic sectors and ecosystem water requirements. It can be significant enough to counterbalance the positive effects of impounding water and offset the contribution of water conservation and reuse practices. Neglecting the critical impact of reservoir evaporation could result in serious overestimation of water availability and therefore underestimation of the required storage capacity to support the desired water release.

All these findings emphasize the need for more studies on basin or regional scale, and even on individual large reservoirs, to obtain more refined assessments and predictions of these losses. While available literature in this field can also help in forming the basis of such future work; few of such studies can be cited from references such as [109][110][111][112].

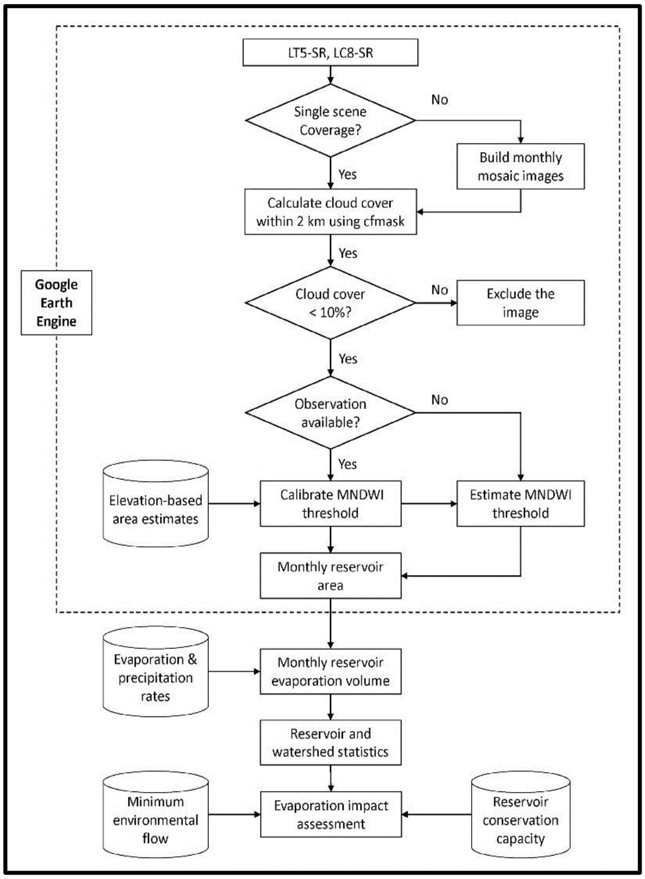

LT5-SR, LC8-SR
Single scene Coverage?
No
Build monthly mosaic images
Yes
Calculate cloud cover within 2 km using cfmask
Yes
Google Earth Engine
Cloud cover < 10%?
No
Exclude the image
Yes
Observation available?
No
Yes
Elevation-based area estimates
Calibrate MNDWI threshold
Estimate MNDWI threshold
Monthly reservoir area
Evaporation & precipitation rates
Monthly reservoir evaporation volume
Reservoir and watershed statistics
Minimum environmental flow
Evaporation impact assessment
Reservoir conservation capacity

Figure 29: Research framework. The abbreviations LTS-Sr and MNDWI stand for Landsat 5 TM surface reflectance, Landsat 8 OLI surface reflectance and modified normalized difference water index, respectively [106].

10. Monitoring Dam Deformation by Remote Sensing

Remote sensing applications have been used not only in following up and evaluating the issues discussed in the previous paragraphs, but, they have been extended to all other related fields, such as climate change impacts on water resources, droughts, floods, and irrigation. One of the important applications has been monitoring the health of dams and water control structures in addition to landslides.

Great advances have been made in the field of remote sensing since the launch of the first satellite by the Soviet Union on the 4th of October 1957. Since then tremendous progress has been made by USA, Russia, EU, China, India in addition to many other countries gradually entering into the space age club. The contribution of USA in this field, however, remains the largest through its (Landsat) and other scientific space programs and the development of a great variety of sensors capable of tracing minute changes on earth surface. Dams and reservoirs have received their fair share of attention through the application of these technologies in monitoring their stability and structural integrity. In recent years, some of these new technologies have been applied to dam deformation measurement, offering unprecedented opportunities to safety management and structural analysis. Among these; Areal Deformation Measurement (ADM) techniques from Terrestrial Laser Scanning and Ground Based (InSAR) and spaceborne (InSAR) sensors which provide the chance to extend the observed region to a large portion of a structure, instead of merely measuring a set of a few control points, in addition to (GNSS) techniques.

Exploitation of satellite-based (InSAR) included the use of variety of; American, Canadian, Japanese, Italian, German, EU satellites dating from 1980s, to the recent European Space agency (ESA) satellites; (Sentinel-1A) and (Sentinel-1B). Together, they have provided (InSAR) coverage on a global scale and on a 6-day repeat cycle [113].

(GNSS) techniques have reached a consolidated maturity for dam deformation monitoring, either for the periodical measurement of networks, or for the implementation into continuously operating systems. Research in the field of data processing is still ongoing, especially for the robust solution of geodetic networks. Eventually, the development of sensor networks and methodologies for data integration has offered the opportunity to analyze different observations in a spatial and temporal content [114].

Monitoring of dams safety issues has always taken a great deal of the dam owners and Engineers attention due to the high risk they present to societies, if neglected. Instrumentation of large dams has been an issue in this kind of high risk civil infrastructures for a long time. The complexity of dams calls for the use of various devices, each of them focusing on a different area of the structure; on structural and non-structural elements, and on different processes such as structural deformations, water infiltration, corrosion, weathering, etc. Since decades, optical instruments and contact deformation sensors have been widely applied to this purpose. These sensors can provide precise observations but limited to a relatively small number of points and with variable time acquisition rate; from quasi-continuous observations up to periodical repetition of observation campaigns at monthly or yearly rate.

The development of new ground based instruments, sensing techniques, and algorithms has opened a new era in the field of terrestrial deformation monitoring and related applications. Robotic total stations with reflector-less range measurement and increasingly also image assistance terrestrial laser scanners[115], and digital photogrammetry allow the acquisition of dense point clouds within a short time, thus yielding detailed snapshot representations of surfaces and a basis for measuring deformations over wide areas. In addition, Ground Based Interferometric Radar (GRIR) can be used to derive differential deformation with very high resolution, in particular with respect to persistent scatters [116].

(GBIR), limitation, however, is its suitability for monitoring small areas from a single building to a slope at short term sampling time up to a few of minutes, as sketched in Figure (30). But the strength of (GBIR) is its complementarily with spaceborne (SAR), Figure (31), the latter is a powerful and effective remote sensing technique for surveying large areas (many km2) at long term (return time in the order of several weeks or months).

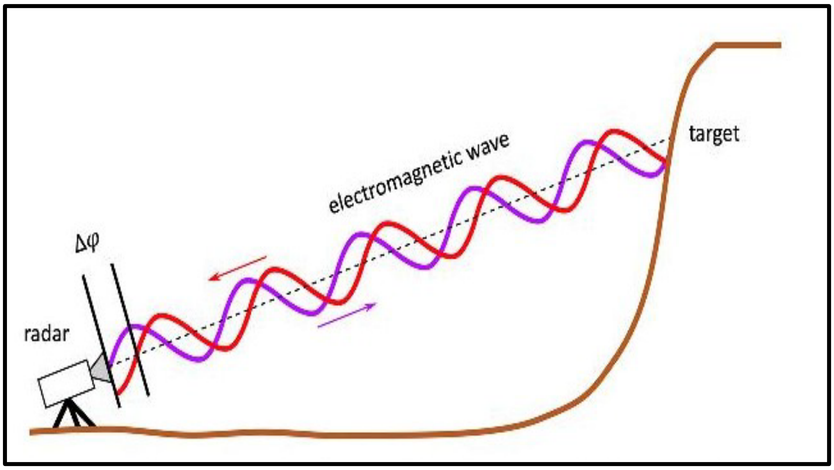


Figure 30: Working Principle of ground transmitter based/ terrestrial radar interferometry [116].

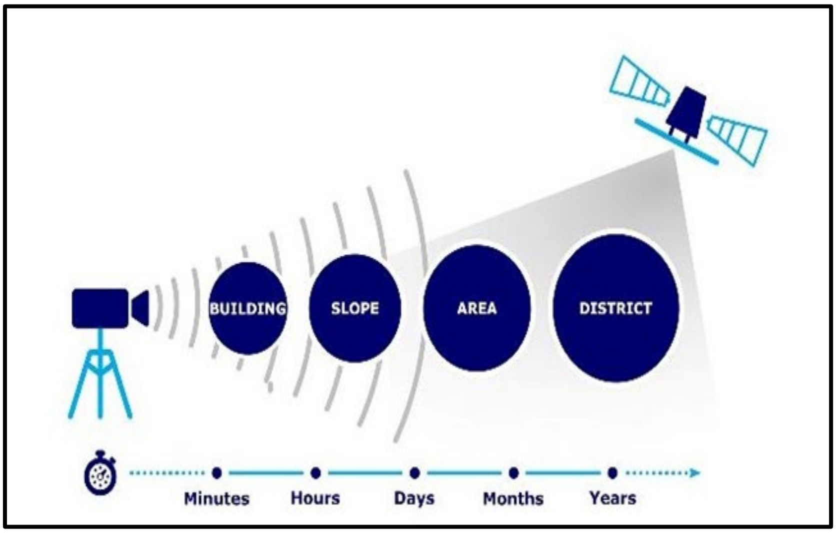


Figure 31: Complementary of GBRI technique with Space Borne InSAR [116].

The amount of data provided by these ground based techniques systems is typically much larger than with traditional geodetic techniques, and they potentially represent a real deformations without a need for generalization from displacements of a few carefully selected control points. However, there is no direct point-to- point correspondence between the point clouds obtained at different epochs and thus entirely new algorithms are required in order to analyze deformations. New concepts for Areal Deformation Monitoring (ADM), namely the (Space-Borne InSAR), and (GNSS) are being developed and investigated including instrumental advances, sensor and process models, and data processing techniques. All of these require thorough validation and assessment before they can be safely applied in critical monitoring applications [117].

One of the (InSAR) techniques developed by an Italian group called (PSP-IFSAR) may be cited for its versatility and varied applications, Figure (32), [118].This technique can provide measurements of displacements due to landslides and other slope failures, dam displacement, hydrogeological changes, tunneling, oil/gas or mineral extraction, earthquakes and volcanic phenomena. It is capable for high volume processing and it has been exploited to allow a nationwide (InSAR) survey of Italy. In the context of its application to dams and reservoirs monitoring, several applications exist as explained below:

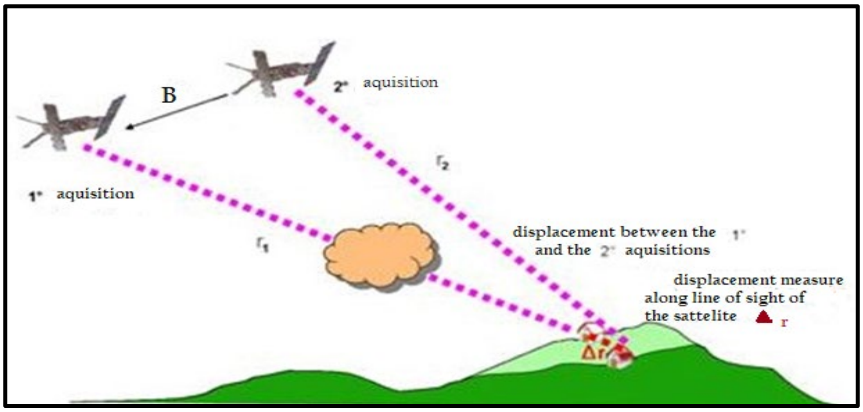

B
2° aquisition
r2
1° aquisition
r1
displacement between the 1° and the 2° aquisitions
displacement measure along line of sight of the sattelite
Δr
r

Figure 32: Principle of InSAR technology [118].

1- Monitoring of the reservoirs surroundings for landslide and other ground displacements; this is particularly effective in exploiting the unique benefits of (InSAR) as it can provide a detailed survey of motion in areas which are often not surveyed at all, or can only be very sparsely surveyed with other methods due to the large surface area. The example presented in Figure (33) of landslide motion on the slopes of Mont Cenis Lake, France, shows such an application where a four-year survey of ground motion depicts the extents of two large landslide bodies. The density of measurements made, with no in-situ installation, varied due to vegetation cover but the availability of useful displacement measurements was remarkable and gave excellent indication of areas of concern where ongoing monitoring via (InSAR) or real-time in-situ monitoring systems could be continued.

2- Measurement of displacements on dam structures themselves: Most dam types lend themselves well to (InSAR) motion measurement. Although in-situ and real time systems are best used for dam safety monitoring, however, (InSAR) can add significantly to the knowledge since historical archives of applicable (SAR) satellite data exist from 1992 to present allowing a retro-analysis of past ground motion to be carried out. This allows dam owners to understand past motion on their dams if no preexisting system existed. This is particularly useful in the cases of a mine tailings dams and other dams which may pass from one owner to another, and where the past knowledge of the dam might be scarce.

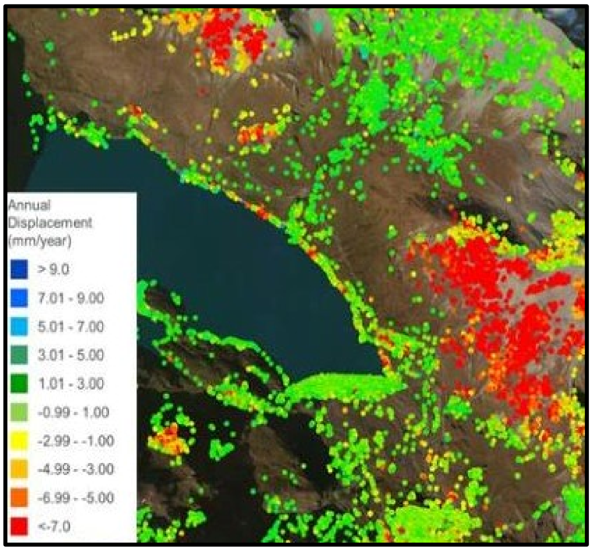


Figure 33: Landslide motion measured by PSP- IFSAR (2011- 2014) on the slopes of Mont Cenis Lake, France [118].

The (GNSS) systems are similarly used worldwide, whereby they lend themselves to position determination in a wide range of applications. Vehicle navigation to asset tracking, i.e. the American (GPS) system, is being the most common of them. These systems are not known for their great accuracy (Particularly about 3m), but this accuracy limitation can be overcome by using range measurement between the receiver antenna and, at least, four satellites, to compute the position of the receiver on the earth at a point in time, Figure (34). The error sources include accuracy in the satellite orbit determination, delays of the (GNSS) signal through the atmosphere, satellite and receiver clock and multipath errors. In order to improve the accuracy of the measurements, different techniques can be used. One of them is to use the Differential (GPS) Technique (DGPS) [119], so centimeter-level accuracy can be achieved. To further improve precision, mathematical methods to remove measurement noise can be applied in order to obtain a weighted average between the predicted and the measured position. Of the most common mathematical methods used is Kalman filtering, which is an algorithm that uses a series of measurements observed over time, containing statistical noise and other inaccuracies, and produces estimates of unknown variables that tend to be more accurate than those based on a single measurement alone [120]. It has been demonstrated empirically that for quasi-static points, a millimeter-level precision can be obtained by this way. This level of accuracy is enough for dam motion monitoring in most of the cases.

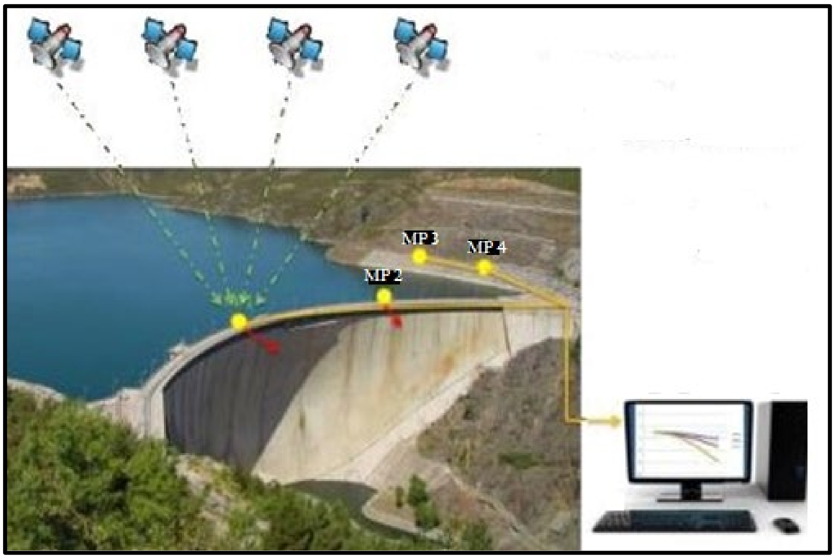


Figure 34: Dam and Slope Monitoring through Differential GNSS. Typical Physical System Schematics [118].

Many dams and reservoirs on which (GPS) with (GNSS) were utilized can be mentioned such as; Pacomica Dam (USA) [121] [122] [123], Libby Dam (USA) [124], La Accena Dam (Spain) [118], El Atazar dam (Spain) also [118].

Among the cases where (InSAR) has been used to monitor dams deformations, some may be cited for illustration of its applicability. The first is the case of Mosul Dam, on the Tigris River, in Iraq. This 113 earthfill dam was observed to be suffering since its completion in 1985 from the dissolution of gypsum karst in its foundation and the deterioration of the deep grout curtain with ongoing deformation which means constant danger of the dam failure, a case which means the drowning of almost of 6.1 people [125] [126] [127].

A team of researchers from distinguished American and Italian universities, German research center, in addition to (NASA) and the Italian Space Agency (ASI), carried out a joint research on the ongoing instability of the dam and presented their work in a paper published in December 2016 [128].

In this study a multi-sensor cumulative deformation map for the dam was obtained, Figure (35), which was generated from space-based Interferometric SAR measurements from the Italian constellation of satellites (COSMO-SkyMed) [129] and the European sensor (Sentinel-1a) [130] over the period 2014–2016. These were compared to an older dataset spanning 2004–2010 acquired from the European Envisat satellite [131]. The study revealed that deformation was rapid during 2004–2010, slowed in 2012–2014 and increased from August 2014 when grouting operations stopped due to the temporary capture of the dam by the self-proclaimed Islamic State (ISIS). The data from these multiple (SAR) satellites was used to measure the deformation at the dam in great spatial and temporal detail which shed new light on the dynamics of the ongoing destabilization.

Data required for the study was obtained from processing 62 images acquired in an ascending track geometry from the Italian Space Agency (ASI) COSMO-SkyMed (CSK) constellation of four satellites to image ground deformation during 2012–2015, and 32 images acquired in a descending geometry by the European Space Agency (ESA) (Sentinel-1A) sensor starting October 2014 and spanning 18 months. Six Interferograms from ESA's (Envisat) satellite, from both ascending and descending geometries, were computed for monitoring the destabilization process during the period 2004–2010. A multi-temporal Interferometric processing technique was applied to the (CSK) and (Sentinel-1A) phase measurements to compute two separate time-series of deformation exploiting point-like scatterers. Time series of the water level at the dam wall using (SAR) amplitude images was then estimated to measure the horizontal distance between the water and the top of the dam and calibrate the results with satellite altimetry data. Data from the different spacecrafts showed that Mosul dam subsided throughout the observation period at varying rates and revealed a shift in the location of the maximum deformation magnitude.

The obtained deformation pattern and magnitudes in the form of subsidence and horizontal displacements are shown in Figures (35), (36) and (37) for the two mentioned periods. Time series of subsidence at the center of the dam is shown in Figure (38). Comparison of the 2004–2010 and 2014–2015 subsidence-rate profiles indicates a 300m eastward shift of the peak subsidence toward the dam's main spillway, Figure (37). The inferred deformation was modeled using a Markov chain Monte Carlo approach to solve for change in volume for simple tensile dislocations. Results from the 2004–2010 and 2012–2014 geodetic datasets suggested that the volume dissolution rate remained constant when the equivalent volume of total grout injected during re-grouting operations was included in the calculations.

The important conclusions of the study may be summarized by:

1- Mosul dam is subsiding at a linear rate of ~15mm/year compared to 12.5mm/year subsidence rate in 2004-2010. The subsidence restarted at the end of 2013 after re-grouting operations stopped. The causes of the observed linear subsidence process can be found in the human activities that have promoted the evaporate subsidence development, primarily in gypsum deposits, and may enable, in case of continuous regrouting stops, that unsaturated water of flow through or against evaporates deposits increases allowing the development of small to large dissolution cavities. Large vertical movements have resulted from the dissolution of extensive gypsum strata previously mapped beneath the Mosul dam. Increased subsidence rate has been due to the absence of regrouting underlying the dam basement. The subsidence seems to have a linear behavior but, a future acceleration due to the increased dissolution speed cannot excluded.

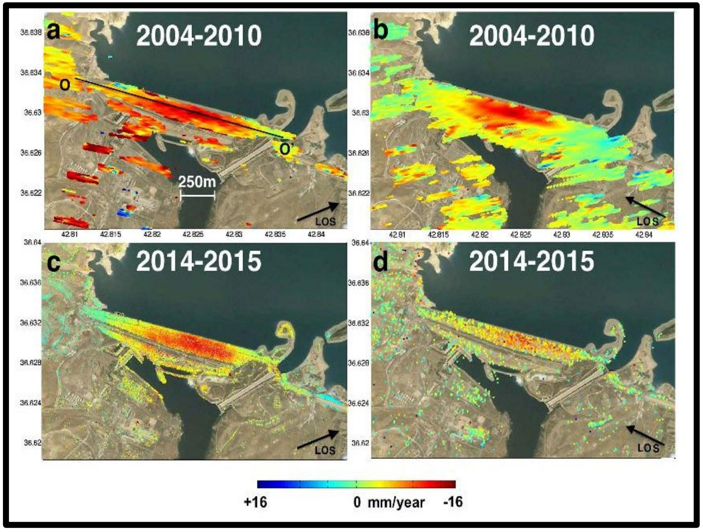


Figure 35: InSAR measured subsidence rates on the Mosul dam, Iraq. Negative values indicate motion away from the satellite, consistent with subsidence. (a) Envisat ascending track DinSAR stacked interfergrams covering May 2004- August 2010. Horizontal/Vertical Profile O-O' is shown in Fig 36(b) Envisat descending track DinSAR interfergrams spanning May 2004- August 2010. (c) CSK ascending track covering December 2014- July 2015 (d) Sentinel descending track covering December 2014- July 2015. Time- series analysis generated using SARPOZ Software [128].

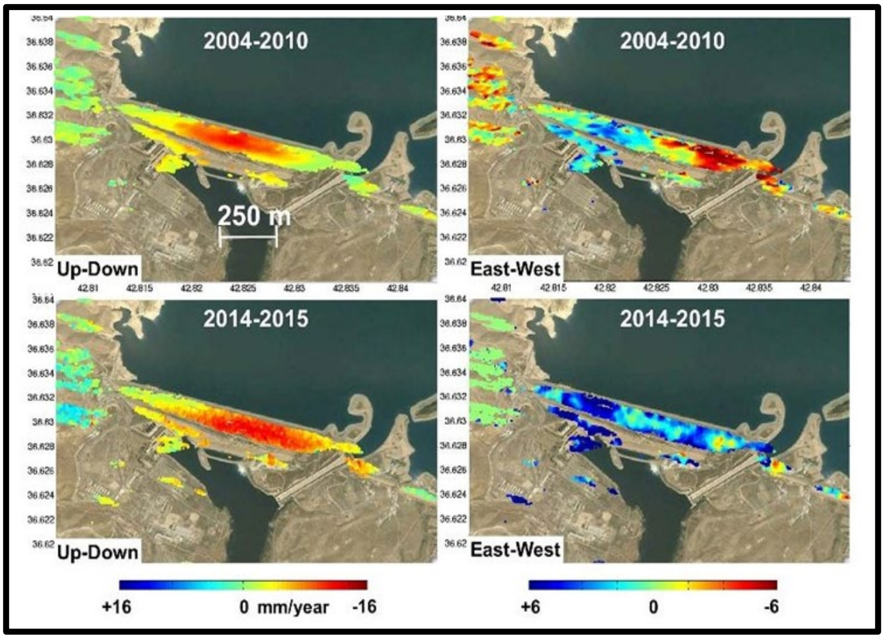


Figure 36. Up-down (left) and east- west (right) displacement at Mosul dam, Iraq. Negative values indicate downward and westward motions, respectively. The data have been resampled on a common grid calculated using the coherence maps from the interfergrams (Envisat) and time-series (CKS- Sentinel 1-a) [128].

2- The slowdown of the deformation can be interpreted as being due to the reduced water level during the December 2012 to June 2013 dry season, Figure (37b), and consequent low water pressure at the dam basement, which results in reduced dissolution rates of the evaporates underlying the dam foundation.

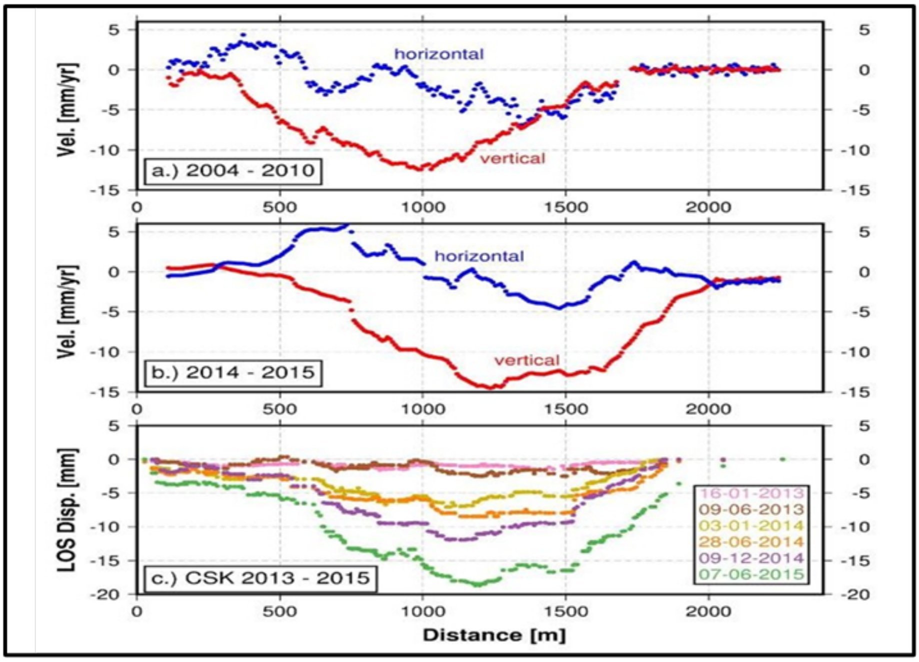


Figure 37: Subsidence profiles. (a) Vertical and east component of surface velocities along the Mosul Dam from Envisat 2004- 2010 2D deformation (b) velocity components from CSK, Sentinel-1a velocity maps obtained using data spanning the same time period (December 2014- July 2015). The results confirm that the vertical velocities greatly exceed the horizontal contribution typical of subsidence patterns, (c) CSK LOS cumulative displacement profiles [128].

3- The inferred lateral propagation of the deformation source seems to be compatible with a hydrogeological model constrained by ground measurements in which the dam substrate is characterized by an increased hydraulic gradient toward the east abutment: promoting fluid pathways connecting the reservoir to the downstream rocks in the subsurface. The model identifies the reservoir-induced pressure as the main cause of an increase in the dissolution rates. Moreover, the abundant presence of readily soluble gypsum below the eastern dam foundation close to the main spillway seems compatible with the observed subsidence shift and the eastward deepening of the dissolution front. These observations agree with the previously mapped geometry of the sedimentary strata, characterized by eastward dipping gypsum beds. SAR data analysis together with the inferred lateral propagation from knowledge of the geology and hydraulic connectivity suggest that the locus of foundation dissolution has shifted eastward toward the dam’s principal spillway.

4- Model Inversions for the locus of volume loss in 2004–2010 infer a depth of 70 meters below the surface and a deepening in 2014–2015 of 160 meters below the surface. The annual increase of water levels in the future, especially during the summer months, could lead to an increase of water pressure in the dam foundation, which combined with the absence of regrouting could speed up the dissolution of the dam substrate and promote its destabilization, Figures (35d) and (38a), extending the observations until March 2016.

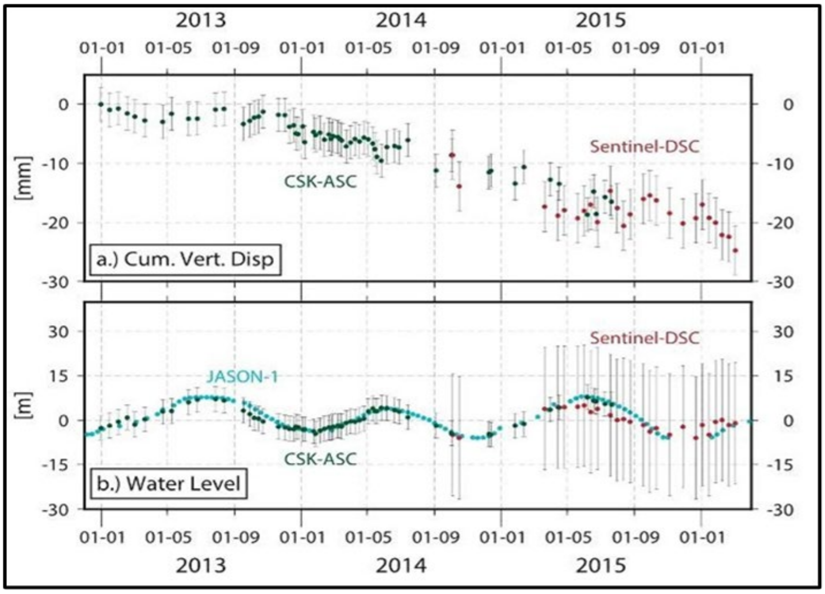


Figure 38: (a) CSK/Sentinel-1a Time series of subsidence at the center of the dam assuming negligible North- South motion (b) CSK/Sentinel- 1a amplitude derived shoreline location on the dam face calculated from amplitude measurements calibrated with JASON -1 altimeter data [128].

5- The Markov chain Monte Carlo (MCMC) approach enabled the calculation of the dissolved volume loss rates at different times which were; 34.1 · 102 ± 9 m3 /year during 2004–2010; vs., 44.6 · 102 ± 9 m3 /year in 2014–2015, indicating a speed up of the dissolution that was likely caused by the lack of maintenance and regrouting.

6- This study highlights how the availability of new constellations of (SAR) sensors supported by historical (SAR) databases are enabling dam deformation monitoring in which (InSAR) measurements are accurate enough to measure sub-kilometer scale geologic and anthropogenic processes and providing critical information fundamental for mitigating anthropogenic hazards.

7- The adopted observational strategy together with simple elastic models agreed with independent studies confirming both the depth of the past dissolution source and the eastward shift of the dissolution front that occurred after the regrouting stopped, [128].

One lecture delivered in the Mosul Dam World Workshop held on 24- 24 May, 2016 in Stockholm gave lesser values for surface settlements and average deformation velocity. The study, however, used only limited number of (InSAR Sentinel 1a), images for the period from 04 October 2014 to 08 February 2016. Compared to the previous study cited above these values may be considered rough and only indicative due to limited data used [132].

In a study published in 2019 [133] the destabilization of Mosul dam was shown to be continuing. This was followed by utilizing again Synthetic Aperture Radar (SAR) as (PS- InSAR) [134] scenes from (Sentinel-1A) for characterizing deformations. Seventy-eight Single Look Complex (SLC) scenes in ascending geometry from the (Sentinel-1A scenes) acquired from third October 2014 to twenty seventh June 2019 were used. A lot of 96 points within the dam structure, were selected to determine the deformation rate using persistent Scatterer interferometry (PSI). Maximum deformation velocity found from the study was about 7.4 $mm \cdot yr^{-1}$ at a longitudinal subsidence area extending over a length of 222 m along the dam axis. The mean subsidence velocity in this area was about 6.27 $mm \cdot yr^{-1}$ and located in the center of the dam. Subsidence rate showed an inverse relationship with the reservoir water level. It also showed a strong correlation with grouting episodes. Variations in the deformation rate within the same year were most probably due to increased hydrostatic stress which was caused by increased water storage in the dam that resulted in an increase in solubility of gypsum beds, creating voids and localized collapses underneath the dam. (PSI) information derived from (Sentinel-1A) proved to be a good tool for monitoring dam deformation with good accuracy, yielding results that can be used in engineering applications and also risk management.

The study demonstrated the following:

i- It demonstrated the successful application of the Persistent Scatterer Interferometry (PSI) approach for mapping unstable areas and determining geo-hazard risks at Mosul Dam.

ii- The area facing the powerhouse, designated subsidence area A, including PSI-1 to PSI-6, had a mean subsidence velocity of about $-6.27\ mm \cdot yr^{-1}$. Area A includes thick gypsum layers, where solutioning activities have created larger voids due to the presence of the Sinjar-Duhok-Amadiya-Fault under the dam foundation. This area also accounts for the maximum hydrostatic pressure due to its lowest topographic elevation in the Mosul Reservoir, causing greater subsidence than elsewhere in the dam.

iii- Subsidence at the old course of the Tigris River i.e., deepest section of the dam, had significantly increased after 2014 due to termination of grouting activities by ISIS, leading to increased dissolution beneath the dam.

iv- In order to maintain safety, the study strongly recommended maintaining the water level at 319 m (a.s.l.) in the Mosul Reservoir. Increasing the water level beyond 319 m will speed up failure of the Mosul Dam. The influence of higher water levels on increase in subsidence was clearly seen during the winter of 2014–2015 and 2016–2017, fall 2015 and 2016, and spring 2016 and 2017.

v- In general, maintaining stability and safe operation of Mosul Dam calls for continuous grouting and a controlled rise of water level in the Mosul Reservoir.

vi- Relevant institutions should explore other ways to store water of the Tigris River to meet various needs. Use of other SAR frequencies such as X-band are also encouraged for further studies [133]

The most recent follow up of the destabilization question of Mosul Dam was given in a study published in July 2020 [135] covering two cases; the first was analysis of the stability of the access bridge on the Tigris River leading to the dam site and, the second which was a more exhaustive analysis which deals with the dam itself, which is more of concern to us. Both cases used Modal and (PS- InSAR) techniques. The specific goal of the second analysis was to identify potential cracks existing on Mosul Dam which could have formed from dams displacements. The method used was based on Modal Analysis of the extended and elastic bodies typical of large infrastructures. Results were also compared with those obtained by (PS- InSAR) techniques applied on data obtained from (COSMO-SkyMed) satellite which belong to the Italian Space Agency [136].

The procedure used can be seen as an in-depth Modal assessment based on the micro-motion estimation, through a Doppler sub-apertures tracking and a Multi-Chromatic Analysis (MCA); the method is based initially on the Persistent Scatterers Interferometry (PSI). The modal analysis detected the presence of several areas of resonance that could mean the presence of cracks, and the results showed that even in 2020 the dam was still in a strong destabilization conditions. Moreover, the dam appeared to be divided into two parts: the northern part was accelerating rapidly while the southern part was decelerating and a main crack in this north-south junction was found.

This study confirmed the results of the previous study of 2019 mentioned above and, the estimated velocities through the PS-InSAR technique showed a good agreement with the (GNSS) in-situ measurements, resulting in a very high correlation coefficient indicating that the procedure works efficiently.

Mosul Dam studies a/m were follow up and in line with research on the use of (SAR) in investigation of dam stability.

In 2016 such work was applied on the Pertusillo arch dam, Basilicata, Italy [137]. The reader may also refer to other papers on the same type of work from referenced such as [138] and [139].

The remote sensing application of other advanced techniques such as the (GNSS) on the deformation and instability monitoring of dams can provide also very good tool in this field. (GNSS) was applied in large number of studies of which we cite few examples documented in number of references such as; 2015 [140], 2016 [141], 2018[142][143][144][145][146][147], 2019 [148], 2020 [149].

11. Summary Notes and Concluding Remarks

Satellite remote sensing is relatively young technology born at the late fifties of the last century, but one which has found its applications in great many fields. Apart from the obvious military uses it has been applied in fields of water resources. In hydrology remote sensing applications were used to estimating hydro-meteorological parameters, such as; land surface temperature, near-surface soil moisture, snow cover, water quality, surface water or sediment accumulations, surface roughness, and land use cover. Moreover, it has been applied to assess fluxes such as evapotranspiration, evaporation from water bodies such as lakes and reservoir, in addition to delineating physiographic variables that can influence hydrologic processes. In Geology remotely sensed data are used to derive fine details of geologic features such as landslides, fault lines and anomalies, mineral explorations and similar fields, and in water resources management it has been used in water resources mapping such as in rainfall- runoff studies, drought monitoring, flood forecasting, irrigation management, rainfall water harvesting, watershed planning and management, groundwater studies and dams and reservoir monitoring and studies. Major questions touched upon in this review are related to dams and reservoirs sites selection, dams and reservoirs management, reservoirs surface area and volume changes, bathymetric surveys and sedimentation assessment in relation with catchment erosion, reservoir water quality follow up, and last but not the least monitoring the behavior of dams and their integrity with the added capacity of looking into this in retrospect.

Remote sensing advances in the last fifty years owes its successes to the great technical innovation in producing sensors that use energy fields, whether in passive or active modes to detect even minute variations on the earth surface and record them in imagery formats. These successes, however, were only possible by using satellites as platforms and development of means to process these images, and algorithms and software to refine the results. Space programs carried out by countries and space agencies around the world have provided multitude of scientific satellites for carrying out this work enhanced by development of specialized satellite-born sensors. Today there is a horde of live satellites and sensor roaming in their selected orbits around the earth, while many others have gone dead after expiring their lives, and there are many more refined ones planned for the future. The satellites used so far or in use today have varied according to the specific objectives they were designed for. To mention only few the (SEASAT), (GEOSAT), (Topex/Poseidon or T/P), (ERS-1), (ERS-2), (GFO), (ENVISAT), (JASON-1) and (JASON-2) are examples of those out of service now which are replaced by other more modern ones such as the American (LANDSAT)s, the Japanese (JERS-1)and (ALOS 2), the Canadian (RADASAT)s, the French (Jason-3), the Chinese (BeiDou) satellites in addition to many Italian, Indian and EU satellites.

Development of sensors technologies has produced some powerful ones such as the Moderate Resolution Imaging Spectroradiometer (MODIS), as well as Synthetic Aperture Radar (SAR), and these sensors have the refined capability of estimating water storage and storage changes in lakes and reservoirs, in addition to measurements of both surface water area and bathymetry and many more variations in the water management fields. In comparison with other satellites, the primary advantage of (LANDSAT) is the high resolution of its sensors but it has low repeat frequency of 16 days and is susceptible to cloud cover contamination. The instruments on the (Landsat) satellites have acquired millions of images which are archived in the United States and at (Landsat) receiving stations around the world presenting unique resource for global change research and applications in agriculture, cartography, geology, forestry, regional planning, surveillance and education, and can be viewed through the U.S. Geological Survey (USGS) "Earth Explorer" website. The latest (Landsat 8) satellite launched on February 11, 2013, has payload consisting of two science instruments; the Landsat8 (OLI) with two bands of Thermal Infrared Sensor (TIRS) (Band 10 & Band 11). These two sensors provide seasonal coverage of the global landmass at a spatial resolution of 30 meters (Visible, Near InfraRed, Short-Wave InfraRed;100 meters (thermal); and 15 meters (panchromatic)[150].

In this review (MODIS) application is exemplified in a comprehensive study on the derivation of water level and volume variations time series of five of the largest reservoirs in USA, in addition to those of 34 more large dam reservoirs in the world, and derivation of the water surface elevation vs. surface area equations for these reservoirs. Such outputs can still be used in further studies, whether on evaporation losses estimates or bathymetric studies or other operational issues. (MODIS) data were used to fill in gaps in recorded historical data of these variables and proved to be a powerful tool in providing the full history of this variation. In other situations, (Landsat) utilizing the Enhanced Thermal Mapper sensor (ETM) borne by (Landsat 7) was used to collect baseline data of small reservoirs surface areas for adopting water conserving policies for such reservoirs scattered over large rural areas located in semi- arid regions where water resources are precious and where collection of such data can be very time consuming and costly. These benefits were further clarified in this review by presenting the results of a study carried out on monitoring three small reservoirs in Ghana which proved that regional to basin scale inventories of small inland water bodies are readily obtainable by using satellite remote sensing. In one case described in this review data was retrieved from the European Satellite (ENVISAT)using advanced (SAR) images.

Creating inventories of small dams and farm ponds on a regional scale using different available satellites imagery outputs is highly recommendable as this can help water management authorities for better water resources management in minimizing water waste and detecting all the unlicensed farm ponds. Accumulating such data base may help also in better water tax collection increasing the irrigation authorities revenue. This type of satellite observation was done for identifying about 1000 small dams and farm ponds in Mzingwane catchment (Zimbabwe).

Such studies with constant follow up can be suggested for countries which suffer from decline of water resources such as Iraq, where there exist thousands of unlicensed fish bonds which encroach on the irrigation projects water shares and discharge their affluent into drains already suffering of high water levels caused by wasteful irrigation practices. The development of the use of satellite remote sensing in bathymetric studies is described in the review by presenting two studies which shows that such methods can be used on large and small reservoirs, but with certain limitations. With the contribution of (ICESat-2) the obtained dataset has the potential to be expanded to thousands of reservoirs and lakes in the future.

The value of remote sensing applications is further investigated in the present review by presenting the use of these applications in sedimentation studies that linked reservoir sedimentation with erosion rates assessments in their respective watersheds. Such studies were carried out for assessing required watershed protection measures needed and for prioritization of sub-basins in need for such amelioration works. The presented studies are selected from India, Indonesia and Brazil to show the applicability of such methods to various watersheds in different regions of the world where erosion presents serious problems. More of similar studies were carried out in recent years for the assessment of sedimentation of individual dams reservoirs without reference to conditions prevailing in their watersheds and some examples are given here.

In another direction, remote sensing depended on the changes of optical properties of water caused by the presence of various contaminants to assess water quality parameters in water bodies such as lakes and reservoirs. This is touched upon in this the review and supported by real case studies. Water quality parameters are taken to mean all physical, chemical, thermal and biological characteristics of water e.g., temperature, chlorophyll content, turbidity, clarity, total suspended solids (TSS), nutrients, Colored Dissolved Organic Matter (CDOM), dissolved oxygen, pH, Biological Oxygen Demand (BOD), Chemical Oxygen Demand (COD), total organic carbon, and bacteria content. Most of these parameters are found to be amenable to characterization by using the appropriate sensing devices with the use of the fitting wavelength of the EM radiation.

Moreover, thermal remote sensing methods have been widely used to estimate water surface temperature in lakes and estuaries. The use of (Landsat5 - TM) was introduced in the review as this method was used by the USBR in the late 1980s to monitor water quality in its reservoirs. But since then there has been great developments in these applications coupled by remarkable progress in space technology. Examination of the literature shows that the past 10-15 years have brought about a focal shift within the field, where researchers are using improved computing resources, datasets, and operational remote sensing algorithms to better understand complex inland water systems. Future satellite missions promise to continue these improvements by providing observational continuity with spatial/spectral resolutions ideal for inland waters. This progress is outlined and study cases are presented for eutrophication in some reservoirs in USA with description of the imagery and algorithms used indicating limitations of the method. Another study on reservoir water quality assessment in Turkey is also presented to show how different methods using different sensors, which was in this case the (RapidEye) satellite could produce very good characterization of water quality.

Shifting focus from water quality research to the research on water availability questions, this review deals next with water losses from reservoirs due to evaporation. This loss is recognized as one of the major factors that should be taken into consideration in the water management of reservoirs, but one which is not given so far its proper weight, or if this was done then available methods of evaluation are either not fitting or inaccurate. Useful use of remote sensing products derived from (MODIS) for such evaluation is demonstrated by presenting a study carried out on five reservoirs in the Savanah River basin in USA. The aim was to evaluate the seasonal variation in evaporation of these reservoirs and comparing the evaporation rates obtained from Mass Transfer Methods based on (MODIS) with the evaporation loss rates calculated from Pan Evaporation measurements. In this way avoiding the use of the cumbersome Conventional Mass Transfer Methods or Pan-Evaporation measurements which tend to be sensitive to local weather and landscape conditions.

The study utilized data from two sources; the first source was (MODIS) for obtaining spatially resolved lake surface temperature by combining measurements from the (MODIS) sensor on (NASA)’s (Terra) and (Aqua) satellites since these data were available from the (NASA) Earth Observing System Data and Information System from July 2002 to December 2012; and from a second source which was the National Weather Service meteorological stations close to the studied reservoirs, which supplied land temperatures measurements, wind speed, eddy flux and relative humidity. These data could be used to quantify the potential error associated with using generated (MODIS) remote weather data. Likewise, local measurement of lake surface temperature could be used to assess the quality of the (MODIS) data and can be also utilized to validate the algorithm employed for filling in missing (MODIS) data. The researches claim that since this approach requires only an existing satellite resource with global coverage and existing meteorological stations close to the study area and having enough recorded data, so this method can potentially be ported to any lake where there is a nearby meteorology station. Hence, this method could be used by both water resource managers and limnologists alike with the possibility of extending this approach beyond a single basin to encompass an entire geographical region or continent. As we see it, this conclusion, however, has to be verified by other studies on various climatic regions to confirm its absolute validity or discover some limitations.

Another approach is also presented in one more study which proposes remote sensing framework to estimate reservoir evaporative loss at the regional scale. This framework uses a multispectral water index to extract reservoir area from (Landsat) imagery and estimate monthly evaporation volume based on Pan-derived evaporative rates. The optimal index threshold is determined based on local observations to be extended to unobserved locations and periods. Built on the cloud computing capacity of the Google Earth Engine (GEE) this framework can analyze satellite images at large spatiotemporal scales, where such analysis is infeasible with a single computer. The study involved 200 major reservoirs in Texas, captured in 17,811 (Landsat) images over a 32-year period. Results of this study indicate that the magnitude of water loss through evaporation is comparable to water use of major economic sectors and ecosystem water requirements. It can be significant enough to counterbalance the positive effects of impounding water and offset the contribution of water conservation and reuse practices. Neglecting the critical impact of reservoir evaporation could result in serious overestimation of water availability and therefore underestimation of the required storage capacity to support the desired water releases. All these findings emphasize the need for more studies on basin or regional scale, and even on individual large reservoirs, to obtain more refined assessments and predictions of these losses. It is important to note that this method is not developed to estimate evaporation in an area where all reservoirs are totally ungauged. However, this method is capable of generating a satellite-based direct quantification of reservoir area dynamics, completing or extending the current area estimates, expanding the estimates to ungauged reservoirs that are close to gauged reservoirs.

The final part of the present review concentrates on one of the most important applications of remote sensing techniques in the field of water management; which is monitoring dam’s structures to follow any developing problems threating their stability and their integrity. Most dam types lend themselves well to Interferometric synthetic aperture radar (InSAR) motion measurement. Although in-situ and real time monitoring and observation systems are best used for dam safety monitoring, (InSAR), however, can add significantly to the knowledge since historical archives of applicable (SAR) satellite data exist from 1992 to present allowing a retro-analysis of past ground motion to be carried out.

Similarly, the Global Navigation Satellite System (GNSS) which refers to a constellation of satellites providing signals from space that transmit positioning and timing data to (GNSS) receivers can also be used to position determination allowing deformation measurements. Accuracy of the results obtained by (GNSS) can be much improved by different techniques, one of them is to use the (DGPS) Technique, so centimeter-level accuracy can be achieved. To further improve precision, mathematical methods may be applied to refine the accuracy such as Kalman filtering algorithm which removes statistical noise and other inaccuracies, and therefore produces a millimeter-level precision which is enough for dam motion monitoring in most of the cases. As a demonstration of the applicability of these techniques in dam deformation studies the chronic case of Mosul Dam is presented in this review by citing series of studies and one international workshop presentation in which (SAR) imagery was used during 2016- 2020.

Mosul Dam is considered at the moment as the most dangerous dam in the world where the persistent dissolution of gypsum in its foundation is challenging the continuous grouting works that are carried out in a non- stop mode and putting the dam on the verge of failure. This condition has merited the importance attached to it and the attention devoted to its stability leading therefore to the use of the most up to date satellite remote sensing for the observation of its integrity. The deteriorating conditions have been showing in; three dimensional displacements on the dam body, migration of the dissolution zone in deeper formations, increased dissolved volume rates when grouting stopped during 2014-2015 indicating the absolute necessity of continuous maintenance grouting, increased subsidence when maintaining high water levels in the reservoir beyond elevation 319 (a.s.l) which occurred during the winters of 2014–2015 and 2016–2017, fall 2015 and 2016, and spring 2016 and 2017.

The latest study carried out in 2020 showed that the dam was still in a strong destabilization conditions and it appeared to be divided into two parts: the northern part was accelerating rapidly while the southern part was decelerating and a main crack in this north-south junction was found. Moreover, this study confirmed the results of the previous studies. Cleary the results of all these studies should be translated in to management procedures to which the dam management and owner must adhere to. These shall be:

First, grouting works must be continued until permanent economical and technical feasible solution is found.

Second, strong vigilance over the dam stability conditions must be kept all the time by following the enhanced monitoring and observation systems installed in 2016 in accordance with the time schedules and frequencies prescribed.

Third, frequent satellite remote sensing studies must be performed to extend the information gathered by the conventional monitoring systems to discover any missed anomalies and have an overall view of the dam- foundation interaction and dam reactions.

Other cases of using (SAR) techniques are mentioned in this review to show their universal application; such as the Pertusillo arch dam in Basilicata, Italy and references to more cases are given to other similar cases. The remote sensing application of (GNSS) as one more advanced technique used for the deformation and instability monitoring of dams is touched upon lightly in the review, but the reader is referred to ten cases as supplemental reading and a list is given of the references.

Generally speaking, it may be concluded that in spite of the large volume of research done on this subject so far, of which this review cites some, the expected future developments in remote sensing technology coupled with advances in algorithms and models used in refining satellite imagery and validate the results will bring more accurate result and less laborious treatment work.

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